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45.1 GENERAL

The purpose of rating a bridge is to determine the maximum vehicle weight a bridge can safely carry. It would be convenient if some simple measure such as gross vehicle weight could be used to determine a bridge's capacity. However, the actual capacity depends not only upon the gross weight, but also upon the number and spacing of the axles and the distribution of load between the axles. A bridge that can carry a given load on two axles can generally carry a larger load spread over several axles. Since it is not practical to rate a bridge for the almost infinite number of axle configurations of trucks, tractors, and trailers on our highways, bridges are rated for standard vehicles which are representative of the actual vehicles on the highways.

Whenever a bridge on the State Trunk Highway System is not able to safely carry the loads allowed by State Statute, they are posted for their capacities. State Statutes allow a gross vehicle weight of 80,000 pounds. Loads up to 167,000 pounds are allowed for annual permit loads.

Bridges are rated at two different load levels referred to as "Inventory Rating" and "Operating Rating". FHWA requires that the standard vehicle used is the AASHTO HS truck and the HS lane loading. Note that other standard vehicles shown in Section 45.6 are used for bridges that need to be posted; however, they are not used when determining the Inventory or Operating Rating. Ratings are required for all Federal Aid bridges. In addition to the Inventory and Operating Ratings, the "Maximum Vehicle Weight" based on the Standard Permit Vehicle at the Operating Rating level is also required.

The National Bridge Inspection Standard (NBIS) requires that load ratings be in accordance with the AASHTO "Manual for Condition Evaluation of Bridges". The AASHTO Manual provides a choice of load rating methods. The methods include the "Load and Resistance Factor" (LRFD) rating method and "Load Factor" method. FHWA has selected the "Load Factor" method as being the most suitable to use as a national standard. All ratings reported to the "National Bridge Inventory" (NBI) for new, replacement, or rehabilitated bridges that require re-rating shall be based on the "Load Factor" method of rating. Either method can be used to establish load limits for the purposes of load posting.

Current practice is to use Load Factor Rating for all bridges except for prestressed girders the positive moment Inventory Ratings are based on service conditions (Allowable Stress Design). Positive moment Operating Ratings and both negative moment Ratings of prestress girders are based on Load Factor.

45.2 BRIDGE INSPECTIONS

To determine the strength or load carrying capacity of a bridge, it is necessary to have a complete description of the bridge as built, any modifications since it was built and its present condition. If drawings are not available or are incomplete, they must be reproduced by means of complete measurements taken in the field. The present condition and changes that may occur in the future are determined from field inspections.

Inspection of bridges on the State Trunk Highway Network is performed by trained personnel from the District Maintenance Sections. Engineers from the Central Office may assist in the inspection of bridges with unique structural problems or when it is suspected that a reduction in load capacity is warranted. To comply with the Federal Bridge Inspection Standards, it is required that all bridges on Federal Aid Routes be inspected at intervals not to exceed two years. More frequent inspections are performed for bridges which are posted for load capacity or when it is warranted by their condition.

Local highway maintenance patrols report sources of danger such as broken or damaged members or components, and potential scour around footings or accumulation of debris in river channels.

45.3 CONDITION OF BRIDGE MEMBERS

The condition and extent of deterioration of structural components of the bridge should be considered in the computation of the dead load and live load effects for the capacity when force or moment is chosen for use in the basic rating equation.

The rating of an older bridge for its load-carrying capacity should be based on a recent thorough field investigation. All physical features of a bridge which have an effect on its structural integrity should be examined. Note any damaged or deteriorated sections and obtain adequate data on these areas so that their effect can be properly evaluated in the analysis. Where steel is severely corroded, concrete deteriorated, or timber decayed, make a determination of the loss in a cross-sectional area as closely as reasonably possible. Determine if deep pits, nicks or other defects exist that may cause stress concentration areas in any structural member. Lowering load capacities below those otherwise permitted or other remedial action may be necessary if such conditions exist.

Size, number, and relative location of bolts and rivets through tension members should be determined and recorded so that the net area of the section can be calculated. Any misalignment, bends, or kinks in compression members should be measured carefully. Such defects will have a great effect on the load-carrying capability of a member and may be the controlling factor in the load-carrying capacity of the entire structure. Also, examine the connections of compression members carefully to see if they are detailed such that eccentricities are introduced which must be considered in the structural analysis.

The effective area of members to be used in the calculations shall be the gross area less that portion which has deteriorated due to decay or corrosion. The effective area should be adjusted for rivet or bolt holes in accordance with the AASHTO Design Specifications.

45.4 INVENTORY RATING

The Inventory Rating of a bridge is a measure of its degree of serviceability. AASHTO defines "Inventory Rating" as the load which can safely utilize an existing structure for an indefinite period. It is simply stated as the rating based on AASHTO design specifications but considering current conditions. When using Load Factor Rating, the basic load factors are 1.3 for dead load and 1.3 times 5/3 for live loads.

45.5 OPERATING RATING

Operating Rating is the load carrying capacity of a structure for the Standard AASHTO HS lane or truck loads and has a smaller load factor applied to live load than is applied for Inventory Rating. It represents the maximum safe load carrying capacity of the structure. It is based on the AASHTO design specifications and/or the AASHTO "Manual for Condition Evaluation of Bridges". An exception to the AASHTO specifications is made regarding the distribution factor for continuous concrete slab bridges of 30 foot width or wider. Operating Rating based on negative moment except at the pier location shall be based on a wheel load being distributed over 12 feet. This assumption is a simplified adaptation of distribution factors given in the Ontario Bridge Design Code. Operating Rating is based on load factors of 1.3 for dead and live load.

45.6 BRIDGE POSTING

A bridge should be capable of carrying a minimum gross live load weight of three tons at Inventory or Operating level. When deciding whether to close or post a bridge, consider the volume of traffic, the character of traffic, the likelihood of overweight vehicles and the enforceability of weight posting. Bridges not capable of carrying a minimum gross live load weight of three tons at Operating level must be closed.

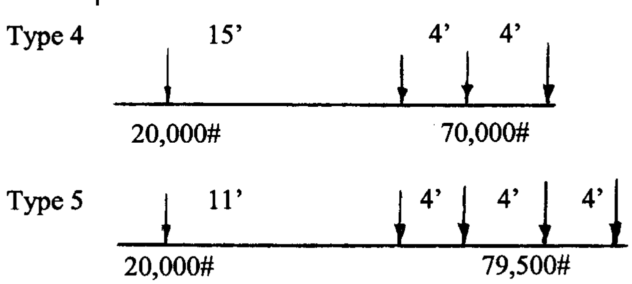
A concrete bridge need not be posted for restricted loading when it has been carrying normal traffic for an appreciable length of time and shows no distress. This general rule may apply to bridges for which details of the reinforcement are not known. However, until such time as the bridge is either strengthened or replaced, it should be inspected at frequent intervals for signs of distress. In lieu of frequent inspections, a bridge may be load tested to determine its capacity.

Bridges which cannot carry the "Maximum Weight by Statutes" shown in Figure 3 for the AASHTO HS20, Type 3S2 or Type 3 vehicles using Operating Rating criteria are posted with one of the standard signs, shown in Figure 1 showing the bridge capacity for the governing vehicle, which should conform to the requirements of the "Manual for Uniform Traffic Control Devices (MUTCD)". A Combination Vehicle is any vehicle with a tractor-trailer.

The State Bridge Maintenance Engineer has the authority to post a bridge and must issue the approval to post any state bridge. State bridges are posted for only one capacity. For exceptional cases local bridges may be posted with the signs shown in Figure 2 using the H20, Type 3 and Type 3S2 vehicles shown in Figure 3.

Since the Maximum Vehicle Weight by "Annual Permits" can be 170,000 pounds, all bridges with a Standard Permit Vehicle rating of 120,000 pounds or less should be posted. This will allow Statutory loads but prohibit Annual Permit Loads. Applicants for Annual Permit Loads over 120,000 pounds are given a list of bridges they cannot cross as the Standard Permit Vehicle Rating is less than their permit load.

In certain cases it may be necessary to post a bridge using the Standard capacity sign shown in Figure 1 for the following trucks which are milk trucks or trash haulers with annual permits.



WEIGHT LIMIT 10 TONS	BRIDGE CLOSED

FIGURE 1 - STANDARD SIGNS USED FOR POSTING BRIDGES

WEIGHT LIMIT 2 Axle Vehicles 15 TONS 3 Axle Vehicles 20 TONS Combination Vehicles 30 TONS	WEIGHT LIMIT 2 Axle Vehicles 14 TONS 3 Axle Vehicles 18 TONS Combination Vehicles 28 TONS	WEIGHT LIMIT 2 Axle Vehicles 12 TONS 3 Axle Vehicles 16 TONS Combination Vehicles 24 TONS

FIGURE 2 - STANDARD SIGNS FOR POSTING LOCAL BRIDGES WITH EXCEPTIONS

Live Loads

The standard vehicles used to determine the required posting of a bridge based on statutory loads are the AASHTO Type H20, HS20, Type 3, and Type 3S2. AASHTO lane loading is not used. The bridge is posted for the lowest weight limit of the standard vehicles.

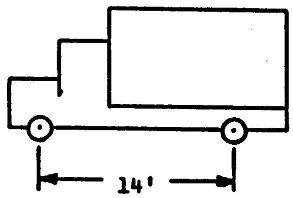
For spans over 200 feet in length the selected vehicle load should be spaced with 30 feet clear distance between vehicles to simulate a train of vehicles in one lane. A single vehicle load should be applied in the adjacent lane(s).

Multiple lane distribution factors using Operating Load Factors are used for determining bridge capacities for posting and annual permits for bridge widths 18 feet or larger. Single lane distribution factors are used for bridge widths less than 18 feet.

Type of Equipment	Standard Vehicles

A. AASHTO WEIGHT B. MAXIMUM WEIGHT BY STATUTE C. MAXIMUM WEIGHT BY ANNUAL PERMIT

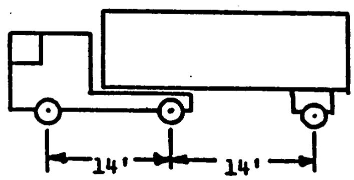
H20



Axle or Axle Group Weights

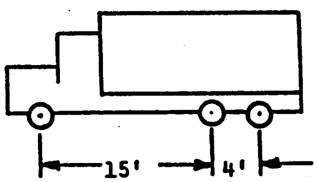
A.	8,000#	32,000#
B.	20,000	20,000
C.	20,000	30,000

HS20



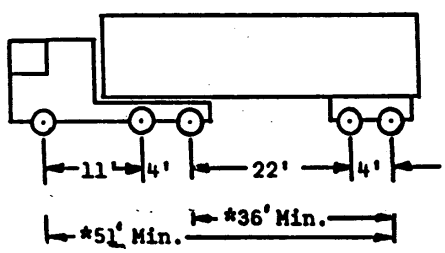
A.	8,000	32,000	32,000
B.	17,100	20,000	20,000
C.	20,000	30,000	30,000

TYPE 3



A.	16,000	34,000
B.	16,500	34,000
C.	21,500	60,000

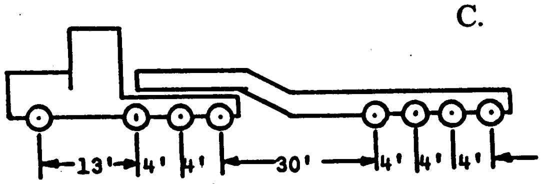
TYPE 3S2



A.	10,000	31,000	31,000
B.	11,500	31,000	31,000
* B.	12,000	34,000	34,000
C.	10,000	55,000	55,000

* Axle spacing for maximum.

STANDARD PERMIT VEHICLE



B. Over 55 foot length. Vehicle not allowed.

C. 17,000 71,400 81,600

FIGURE 3 - STANDARD VEHICLE CONFIGURATIONS USED TO DETERMINE THE LOAD CARRYING CAPACITIES OF BRIDGES

45.7 MATERIAL STRENGTH AND ALLOWABLE STRESSES

Allowable stresses shall be as stated in AASHTO, “Manual for Condition Evaluation of Bridges” or as stated in this chapter. For situations not covered, the AASHTO “Standard Specifications for Highway Bridges” shall be utilized.

The AASHTO "Interim Specifications Bridges 1974" made a number of significant changes in the design of reinforced concrete. Load factor design, bar steel development lengths and elimination of the old bond stress concept are some of the changes. One reason for these changes is to make sure that an overloaded concrete structure fails by yielding of the reinforcing in bending and not by a sudden concrete shear or bond failure. Any inadequacies of concrete members designed prior to 1974 may be due to shear or bond capacity and not bending capacity.

(1) Reinforcing Steel

The following are the allowable unit stresses for tension for the Allowable Stress Method and yield strengths for the Load Factor Method. When the condition of the steel is unknown, they may be used without reduction.

Reinforcing Steel Grade	Inventory Rating (psi)	Operating Rating (psi)	Minimum Yield Point
Structural or Unknown Grade	18,000	25,000	33,000
Grade 40 (Intermediate)*	20,000	28,000	40,000
Grade 60	24,000	36,000	60,000

* Wisconsin started to use Grade 40 bar steel about 1955-1958; this should be noted on the plans.

(2) Concrete

The following are the maximum allowable unit stresses in concrete in pounds per square inch.

Compression due to bending:

Year Built	Inventory Rating	Operating Rating	Compressive Strength (f'c)	N*
Before 1959	1000	1500	2500	12
1959 and later	1400	1900	3500	10
For all non-prestressed slabs 1975 and later	1600	2400	4000	8
Prestressed girders before 1964 and all prestressed slabs	2000	3000	5000	6
1964 and later for prestressed girders	2400	3000	6000	5

* N = modular ratio = E_s/E_c

The "Year Built" column may be used if concrete strength is not available from the structure plans.

(3) Prestressed Steel Strands

Uncoated Seven-Wire Stressed-Relieved and Low Relaxation Strand

Year Built	Grade	Nominal Diameter of Strand-inches	Nominal Steel Area of Strand	Yield Strength	Breaking Strength
Prior to 1963	250	7/16 (0.438)	0.108	213,000	250,000
Prior to 1963	250	1/2 (0.500)	0.144	212,500	250,000
1963 to present	270	1/2 (0.500)	0.153	229,000	270,000
1973 to present	270 Low Relaxation	1/2 (0.500)	0.153	242,500	270,000

The "Year Built" column is for informational purposes only. The actual diameter of strand and grade should be obtained from the structure plans. If an option is given on the structure plans to use either stress relieved or low relaxation strand, or 7/16 of 1/2 inch diameter strand, consult the shop drawings for the new structure to see which option was exercised. If the shop drawings are not available, assume the option which gives the lowest operating rating was used.

(4) Structural Steel

The AASHTO "Manual for Condition Evaluation of Bridges" gives allowable stresses for steel based on year of construction or known type of steel. For newer bridges refer to

AASHTO design specifications.

When rating rehabilitated bridges, they should be rated assuming composite action if they were designed for composite action or shear connectors such as studs or angles are spaced at 2'-0 or less.

(5) Timber

The Inventory unit stresses should be equal to the allowable stresses for the stress-grade lumber given in the AASHTO Design Specifications. Allowable inventory unit stresses for timber columns should be in accordance with the allowable provisions of the AASHTO Design Specifications.

The maximum allowable Operating unit stresses should not exceed 1.33 times the allowable stresses for stress-grade lumber given in the current AASHTO Design Specifications. Reduction from the maximum allowable stress will depend upon the grade and condition of the timber and should be determined at the time of inspection. Refer to the Manual for Condition Evaluation of Bridges for rating timber columns.

The AASHTO Design Specifications allow adjustment to the allowable shear stresses when the length of splits or checks is known.

45.8 ANNUAL PERMIT INFORMATION

Annual permits are only allowable for nondivisible loads such as machines. Farm products, concrete and other construction materials are divisible.

Under Annual Permits the following gross weights are allowed:

Single Axle .... 20,000 Pounds (2 Tires)
Single Axle .... 30,000 Pounds (3 Tires)
2-Axle Tandem .... 55,000 Pounds
3-Axle Tandem .... 70,000 Pounds
4-Axle Tandem .... 80,000 Pounds

A tandem axle is considered to be any group of two, three or four axles in which the centers of successive axles of the group are more than 42 inches and less than 6 feet apart. If the spacing between any combination of single axles or tandem axle groups is less than 18 feet the gross load of the combinations must be reduced. Refer to "Vehicle Weight Authorized by Multiple Trip Permits" available from the Division of Motor Vehicles (DMV).

There is a length limitation of 50 feet for single vehicles and 75 feet for vehicle combinations.

The percentage of allowable gross vehicle loads on the single or tandem axle groups is obtained from the DMV table. For example, for the axle group combination of one single-axle and one 3-axle tandem with "D" = 12 feet, the gross weight is 91,000 pounds. The maximum allowable of 70,000 pounds is assumed to be on the 3-axle tandem. 21,000 pounds or 23.1% of the gross is on the single-axle.

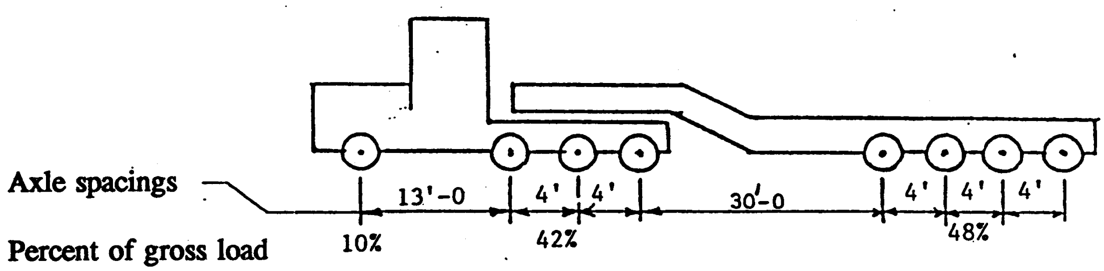
45.9 SINGLE TRIP PERMIT INFORMATION

Nondivisible loads which exceed the annual permit restrictions may be moved by the issuance of a single trip permit.

Tables similar to those used to determine allowable loads for annual permits, but with larger allowable loads, are used to determine the maximum loads that may be moved under a single trip permit.

The holder of an annual permit may move loads, subject to the restrictions for annual permits, over any state highway. When a single trip permit is issued, the applicant is required to indicate on the permit the origin and destination of the trip and the highways that are to be used. Another permit is needed for local roads.

All Federal Aid bridges are rated to determine the maximum weight they can carry on a Standard Permit Vehicle. The rating is the total vehicle weight given to the nearest 10 kips. The maximum rating currently shown on the plans and in the Bridge File is 250 kips.



STANDARD PERMIT VEHICLE

This Standard Permit Vehicle represents the truck most frequently used to carry loads requiring a single trip permit. However, each bridge on the route is analyzed for the vehicle shown on the single trip permit.

Load distribution is based on single lane loading and the distribution factors given in AASHTO 3.23. Single Lane distribution is used as these Permit Loads are infrequent and are probably the only load on the structure in most cases. For continuous concrete slab bridges of 30 foot width or more wheel loads are distributed over a width of 12 feet, which is a simplified adaptation of the distribution factor in the Ontario Bridge Design Code. Allowable stresses or load factors are as specified for operating rating. In special cases impact may be neglected provided that the maximum vehicle speed does not exceed 5 MPH on the bridge.

For bridges with roadways greater than 30 feet the rating for the Standard Permit Vehicle shall be based on the interior girder. For bridges with roadways 30 feet or less the rating shall be based on the minimum value of the exterior or interior girders.

Single trip permits with a gross vehicle weight of 150,000 pounds or more are reviewed by the Bridge Office to determine if the bridges on the selected routes can carry the permit load.

	← D →		← D →		← D →		← D →		← D →	
	o	o	o	oo	o	oo	o	ooo	oo	oo
Axle Spacing	2 or 4 tires	2 or 4 tires	2, 4, 8 tires	4 or 8 tires	2, 4, 8 tires	16 tires	2, 4, 8 tires	12 tires	4 or 8 tires	4 or 8 tires
18' +	70,000		100,000		105,000		115,000		130,000	
17'	70,000		98,000		103,000		113,000		126,500	
16'	69,000		96,000		101,000		111,000		123,000	
15'	69,000		94,000		99,000		109,000		119,500	
14'	68,000		92,000		97,000		107,000		116,000	
13'	68,000		90,000		95,000		105,000		112,500	
12'	67,000		88,000		93,000		103,000		109,000	
11'	67,000		86,000		91,000		101,000		105,500	
10'	66,000		84,000		89,000		99,000		102,000	
9'	66,000		82,000		87,000		97,000		98,500	
8'	65,000		80,000		85,000		95,000		95,000	
7'	65,000		80,000		83,000		93,000		92,500	

Notes: (1) Weights shown apply whether axles are on truck, tractor, or trailer or both.

(2) Reduced weights given above may be further reduced because of limitation of approximately 800 pounds per inch of manufacturer's rated tire width.

FIGURE 4 - SINGLE TRIP GROSS WEIGHT REDUCTIONS FOR AXLE SPACINGS LESS THAN 18 FEET

	|← D →| oo oo		|← D →| oo oo		|← D →| oo ooo		|← D→| oo ooo		|← D →| ooo ooo	
Axle Spacing	4 or 8 tires	16 tires	16 tires	16 tires	4 or 8 tires	12 tires	16 tires	12 tires	12 tires	12 tires
18' +	135,000		140,000		145,000		150,000		160,000	
17'	131,000		135,000		141,000		146,000		156,000	
16'	127,000		130,000		137,000		142,000		152,000	
15'	123,000		125,000		133,000		138,000		148,000	
14'	119,000		120,000		129,000		134,000		144,000	
13'	115,000		115,000		125,000		130,000		140,000	
12'	111,000		110,000		121,000		126,000		136,000	
11'	107,000		105,000		117,000		122,000		132,000	
10'	103,000		100,000		113,000		118,000		128,000	
9'	99,000		95,000		109,000		114,000		124,000	
8'	95,000		90,000		105,000		110,000		120,000	
7'	91,000		90,000		101,000		106,000		116,000	

FIGURE 4 (CONT.)

45.10 PROBABLE MODES OF FAILURE

By test loading full size bridges with loads causing failure and by studying bridges which have accidentally collapsed due to overloads it is sometimes possible to isolate the most probable modes of failure of different bridge types.

The AASHTO road test in 1962 tested both composite and noncomposite steel wide flange bridges, composite prestressed bridges (both pretensioned and post-tensioned), and a reinforced concrete bridge of monolithic T-beam construction. All of the bridges were simple-span. The following generalizations about the probable modes of failure are from the results of the AASHTO road tests and the tests by the North Dakota State Highway Department.

A. Noncomposite Steel Wide Flange

Plastic hinges form at mid-span or at the ends of cover plates. Yielding occurs both on the tension and the compression side. Large permanent deformations occur at loads not greatly in excess of the yield.

B. Composite Steel Wide Flange

Plastic hinges form at mid-span or at the ends of cover plates. Tensile yielding occurs in the bottom flange and in the web to the underside of the top flange. Tension cracking may appear on the bottom surface of the slab even when no signs of crushing appear on the top surface. The transition from elastic deformations to deformations near failure load is very gradual so that even relatively large increases of load beyond that causing first yielding results in only small permanent deformations. This is in marked contrast to noncomposite bridges. For a composite bridge, the magnitude of the yield load is a less critical quantity than for a noncomposite bridge.

C. Prestressed Concrete Bridges

After a certain overload, a decrease in stiffness results due to an increase in concrete tension cracking. After repeated overloads, permanent deflections occur at mid-span caused primarily by inelastic deformations of the prestressing steel. Further deflections occur after possible bond failure of the prestressed strand or a bond failure between the slab and girder starting at the 1/3 points and working toward mid-span. Collapse occurs with actual breakage of the strands. Heavy crushing of the concrete slab will probably occur prior to collapse.

D. Reinforced Concrete T-Girders

The ultimate moment capacity of the beam develops at mid-span and the beam fails by yielding of the reinforcing steel. Inclined cracking develops near mid-span and extends almost to the top surface of the slab. Large permanent deflection occurs prior to collapse.

Crushing of the concrete slab will probably occur just prior to collapse.

E. Reinforced Concrete Slab Bridges

A test of a through-span continuous flat slab was performed by the North Dakota State Highway Department. Unlike the AASHTO road tests where actual vehicles were used to load the bridges to failure, the North Dakota program used a line load across the entire bridge width applied by hydraulic rams. These tests indicated that the collapse load for the bridge can be accurately predicted by considering the formation of yield moments along the centerline and over the piers. Curbs and parapets that are acting structurally with the deck should also be considered.

Other pertinent information obtained from this test follows:

1. The load causing the yield stress in the steel can most accurately be determined by using ultimate strength analysis.

2. A load beyond the load causing a yield stress results in a permanent deflection in the slab.

3. To produce a permanent deflection in the tested bridge, it was necessary to place a load of four times the HS20 truck load in each lane.

4. To produce a collapse of the tested bridge, it was necessary to place a load of ten times the HS20 truck in each lane. The bridge was designed for two lanes of traffic and an AASHTO HS20 truck using AASHTO 1957 specifications.

F. Trusses

Most of the bridges that accidentally fail to service are trusses. Unfortunately, the actual cause of failure after total collapse is often not readily recognizable. Detailed investigations are seldom made because of the urgent need to replace the failed structure and reopen the road.

However, the fact that trusses have a higher rate of failure than other types of structures tells a great deal about the probable cause of these failures. Trusses are made up of tension and compression members. For the tension member, there is a definite factor of safety with no reserve. When the yield stress of the tension member is reached, it fails. Truss compression members fail by buckling and when the load causing buckling is reached, failure also occurs. Since all members of a truss are needed for a truss system to function, failure of any member generally means total collapse. Trusses cannot form the plastic moments that beam or slab bridges are capable of developing to supply an extra reserve and give no advance warning that failure is eminent. Exercise caution when computing the section properties to be used for rating trusses. Deterioration or corrosion of members must be accurately known and, if not accurately know, a conservative assumption of the section loss should be made.

Connections may control the capacity of a truss if the connection elements are weakened by corrosion. The web of floor beams is sometimes all but nonexistent because of severe corrosion. Rivet heads are sometimes popped or bolts may be missing.

Tests to failure have been conducted upon pony trusses. The top compressive chord of the pony truss is generally assumed to be unsupported or unbraced for its entire length. However, tests have shown that the top chord receives some lateral support from the verticals if they are rigidly connected to the floor beams. This results in a failure load above the theoretical load computed by assuming the top chord to be unbraced. The engineer may consider this when selecting a factor of safety for pony trusses.

G. <u>Tests by Burdette and Goodpasture</u>

The tests by Burdette and Goodpasture were performed on the following bridges:

1. Four-span continuous, 36” steel rolled beams, composite in positive moment regions.

2. Simple-span composite with AASHTO Type III (45”) precast, prestressed concrete beams.

3. Simple-span reinforced concrete T-beams, monolithic construction.

4. Three-span continuous, noncomposite, 27” steel rolled beams.

The following comments are supported by information obtained from these tests:

1. The theoretical method employed to compute the ultimate capacity of each of the four bridges assumes that the entire bridge acts as a wide beam spanning between supports. The fact that results predicted by this method were in very close agreement with measured ultimate loads, indicates that all four girders in each bridge attained their ultimate flexural capacity under the same load; that is, the ultimate capacity of a bridge is equal to the sum of the ultimate capacities of the individual girders.

2. Failure to account for the redistribution of moment and all plastic moments that occur in
the limit state in continuous bridges results in a predicted load capacity which is significantly lower than actual. However, it is unlikely that the excess strength developed beyond the load required to develop the first plastic hinge, with its attendant large deflection, is of much practical significance.

3. The load causing “first permanent set” is less readily identifiable than is the “ultimate” load. However, the test results indicate that the requirement in the AASHTO specifications which limits overload in composite beams to 95

percent of the load causing first yield, is a reasonable one. The test results from noncomposite bridges suggest that the limitation of overload to 80 percent of the load causing first yield of noncomposite bridges may be overly conservative.

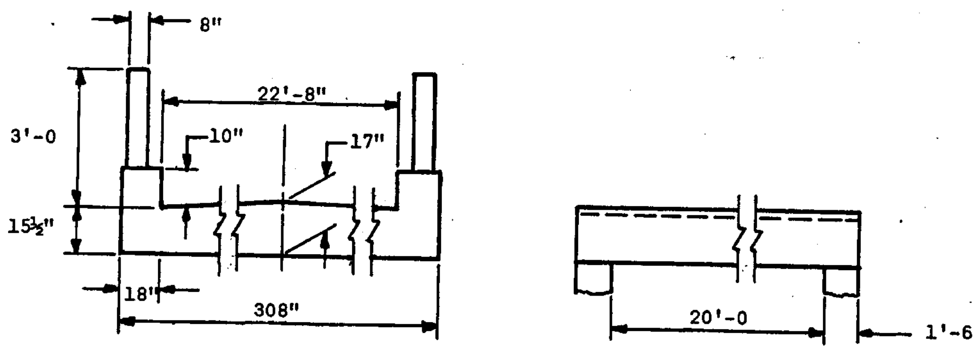
4. The tests clearly indicated that the curbs acted as an integral part of the bridge structures for composite bridges at all load stages up to failure.

45.11 SAMPLE PROBLEMS

(1) Single Span Concrete Slab

The following bridge was built in 1922. Corrosion of the positive steel is small so no deduction in area. There is a 4" bituminous overlay. The parapet is poured the full length with 1/2" square bars at 1'-6" between the parapet and slab. The roadway width is 22'-8 so two lanes are applied.

Area of steel = 69-#7 bars = 41.4 square inches (#7 @ 4 1/2") 1 3/4" CLEAR
fy = 36,000 psi fc' = 2000 psi (From plans)



Assume constant slab depth of 15 1/2 inches to account for the wearing surface deduction.

Live Load Distribution

For inventory and operating rating the wheel load is distributed over a width of

$$E = 4 + .06(S) \qquad S = 20 + 1'{-}6 = 21.5'$$
$$E = 4 + .06(21.5) = 5.29'$$

Moment Capacity - Load Factor Rating

Area of Steel in width $E = (5.29 * 12 / 4.5) * .6 = 8.46\ sq.\ inch$

$$b = 5.29 * 12 = 63.5''$$
$$d = 15.5 - 1.75 - .4375 = 13.31''$$
$$\rho = AS / bd = 8.46 / 63.5 * 13.31 = .0100$$

Compute ρ balanced

$$\rho_b = .85 * .85 * f'_c / f_y ((87{,}000 / 87{,}000 + f_y))$$

$$\rho_b = .04014 * .7073 = .0284$$
$$\rho \langle .75 \rho_b \quad OK$$

Compute design moment strength

$$M = \phi (AS * f_y (d - a / 2))$$
$$a = AS * f_y / (0.85 * f'_c * b) = 2.821''$$

ϕ = strength reduction factor = .9

$$M = .9 * 8.46 * 36 * (13.31 - 1.41) / 12 = 271.8 \quad ft.kips$$

M = design moment strength/wheel

Dead Loads

Slab dead load = [63.5(16.25)/144](.15) = 1.075 kips/ft.
Overlay = .333(.144)(63.5/12) = .254 kips/ft.
Total Dead Load = 1.329 kips/ft.

Since the parapet and curb act structurally and are stiffer than the slab, they support their own dead load when live load is on the structure. Therefore, the dead load of the curb and parapet need not be included.

Dead load moment = 1/8(1.329)*21.5^2 = 76.79 ft. kips

Inventory Rating

M = 1.3 (D + 5/3*RF*(L+I))

RF = rating factor

L + I = live load moment with impact,

From table in Appendix A of AASHTO
Moment for HS20 truck = 172 ft kips without impact.
Impact = 50/(S+125)≤ 0.30; Use 0.3

L + I = 1.3*172 = 223.6 ft. kips

L + I/wheel = 223.6/2 = 111.8 ft. kips

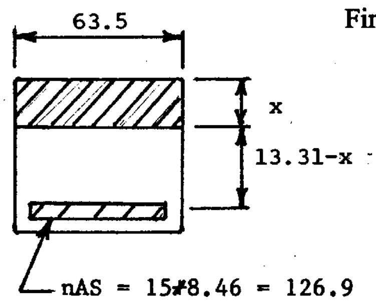
M = 1.3D + 2.17 RF*(L+I)

271.8 = 1.3*76.79+2.17*RF*111.8

RF = (271.8 - 99.8)/(2.17*111.8) = .709

Inventory Rating = .709*HS20 = HS14

AASHTO 8.16.8 can be checked for Fatigue Stress Limits but it rarely if ever governs.



Find neutral axis using transformed area

n = 15

$.5*63.5\, x^2 = 126.9*(13.31-x)$

$31.75x^2 + 126.9x - 1689.0 = 0$

x = 5.56”

$I = 1/3\, bx^3 + nAS(d-x)^2$

I = 3657.8 + 7630.9 = 11289 Inches^4

Steel stress from L + I

$F_s/n = Mc/I$

F_s = 15*111.8*12*7.75/11289 = 13.81 ksi OK

Operating Rating

M = 1.3 (D + RF*(L+I))

271.8 = 1.3*76.79+1.3*RF*111.8

RF = (271.8 - 99.8)/(1.3*111.8) = 1.18

Operating rating = 1.18*HS20 = HS24

Summary

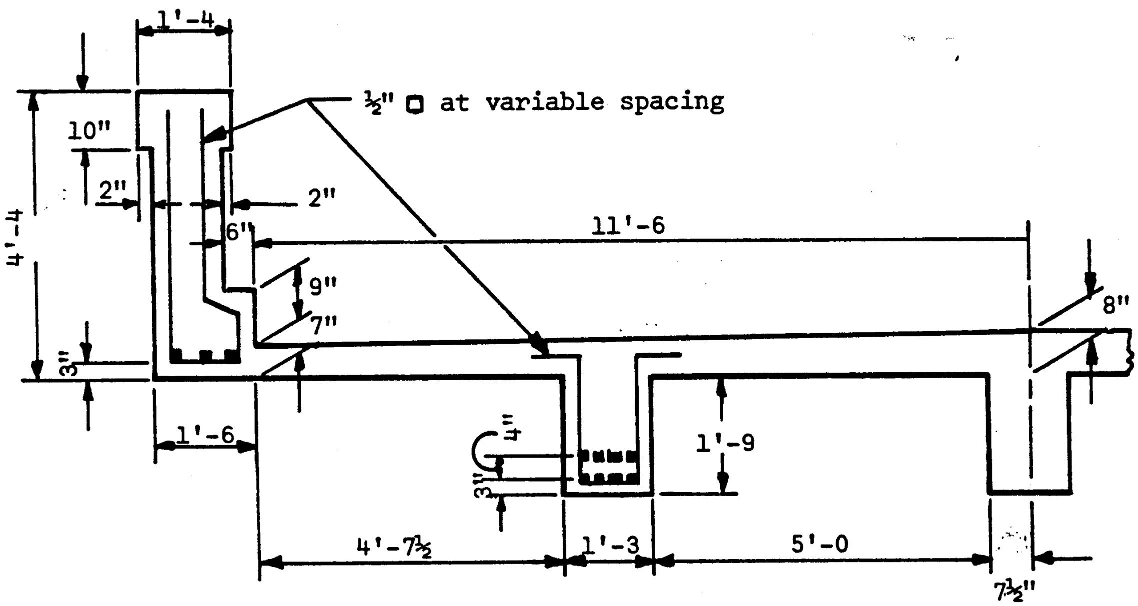
The ratings for this bridge are:

Inventory Rating HS14
Operating Rating HS24

The Standard Permit Vehicle Rating is not required since the bridge is not on the State Trunk Highway Network.

(2) Single Span Concrete T-Girder with Railing Girder (B-20-678)

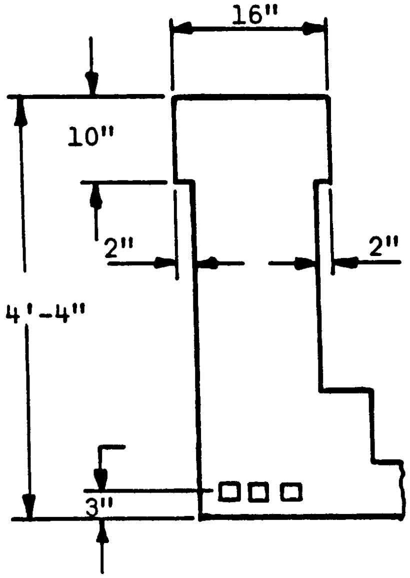
The following bridge was built in 1929 and has a 4" bituminous overlay. The bar steel shows no signs of corrosion. The roadway width is 23' so two lanes must be applied. The parapets on this bridge are designed as exterior girders.



Longitudinal girder steel is 1 1/8" square bars

Clear span is 30'-0, span length equals 31'-6 f'c = 2000 psi, fy = 36,000 psi, N = 15

Exterior Girder Moment Capacity - Load Factor Rating



AS = 3*1.265 = 3.795 Sq. Inches
b = 16 Inches
d = 49 Inches

Computer design moment strength

$M = \phi\ (AS*f_y(d-a/2)$

$a = AS*f_y/(0.85*f_c'*b)$

a = 3.795*36/(.85*2.*16)

a = 5.02

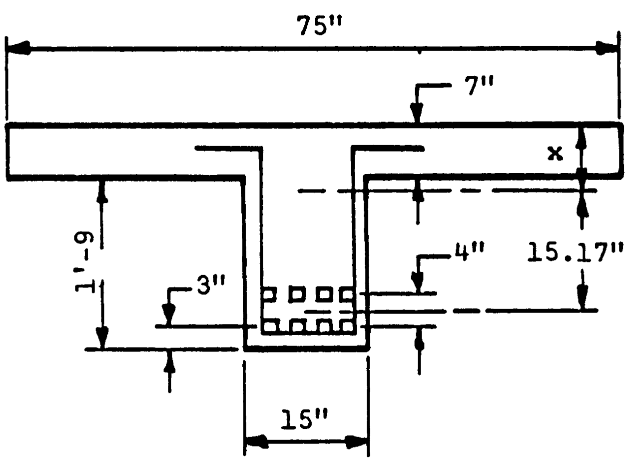
ϕ = strength reduction factor = .9

M = .9*3.79*36* (49.-2.51)/12.

M = 476.4 ft. kips

M = design moment strength

Interior Girder Moment Capacity - Load Factor Rating



AS = 8*1.265 = 10.12 Sq. Inches
b = 75 Inches
d = 21.+7-3-2. = 23 Inches

Computer design moment strength

a = 10.12*36/(.85*2*75)

a = 2.86

M = .9 + 10.12*36.*(23-1.43)/12.

M = 589.4 ft kips

M = design moment strength

Dead Loads

Dead load of exterior girder, slab, and overlay

[1.33(.833) + 1(2.17) + 1.5(1.33)] .15 = 0.791 kips/ft. (girder)

2.3125(.583).15 = .202 kips/ft. (slab)

2.3125(.333).144 = .111 kips/ft. (overlay)

Total exterior girder dead load = 1.104 kips/ft.

Dead load of interior girder, slab, and overlay

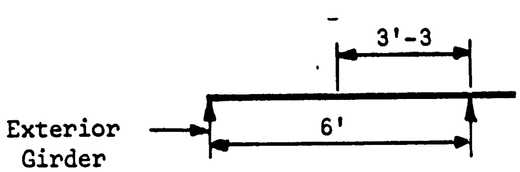
1.75(1.25) + 6.25(.667) .15 = .953 kips/ft. (girder and slab)

6.25(.333).144 = .300 kips/ft. (overlay)

Total interior girder dead load = 1.253 kips/ft.

Inventory Rating

Exterior girder live load moment



Distribution factor for wheel loads

Distribution factor = 3.25/6 = .542
(Wheel placed 2' from curb)

One HS20 truck causes a maximum moment of 304.9 ft. kips without impact.

LL + Impact = 304.9(1.3) = 396.4 ft/kips.

Dead load moment equals 1/8(1.104)$(31.5)^2$ = 136.9 ft/kips.

L + I = 396.4 ft/kips

L + I/wheel = 396.4/2 = 198.2 ft/kips

L + I on exterior girder from an HS20 truck = .542*198.2 = 107.4 ft/kips

M = 1.3(D + 5/3*RF*(L + I))

476.4 = 1.3*136.9 + 2.17*RF*107.4
RF = (476.4 – 178.0)/(2.17*107.4)

RF = 1.28 for exterior girder

Interior girder live load moment

Distribution factor for wheel load = S/6.0 = 6.25/6 = 1.042

L + I = 1.042*198.2 = 206.5 ft/kips

Dead load moment equals 1/8*1.25*31.5*31.5 = 155.4 ft/kips

M = 1.3(D + 5/3*RF*(L + I))

589.4 = 1.3*155.4 + 2.17*RF*206.5

RF = (589.4 – 202.0) / (2.17*206.5)

RF = .864 for interior girder

Inventory Rating = .864*HS20 = HS17

Operating Rating

M = 1.3(D + RF*(L + I))

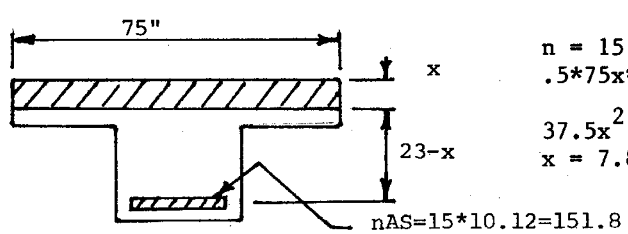
589.4 = 1.3*155.4+1.3*RF*206.5

RF = (589.4 – 202.0) / (1.3*206.5)

RF = 1.443

Operating rating = 1.443*HS20 = HS29

Check AASHTO 8.168 Fatigue Stress Limits



n = 15

.5*75x*x = 151.8*(23-x)

$37.5x^2 + 151.8x - 3491.4 = 0$

x = 7.83

Since x is greater than 7" a T-section exists. The above solution for x assumed the neutral axis fell in the flange. To obtain the exact x value it must be assumed that the neutral axis is in the stem. However, when the neutral axis falls only slightly into the stem, as it does for this girder, the above solution will yield results which are within reasonable accuracy.

$I = 1/3\ bx^3 + nAS(d-x)^2$

I = 12001. + 34933 = 46934

The moment from an HS17 truck = .864*206.5 = 178 ft/kips

Steel stress from HS17 truck

$f_s/n = M_c/I$

f_s = 15*178*12*15.17/46934

f_s = 10.4 ksi < ff

ff = 21 – 0.33 f min + 8 (r/h)

f min = 155.4/178*10.4 = 9.1

r/h = .3

ff = 21 – 3.0 + 2.4 = 20.4

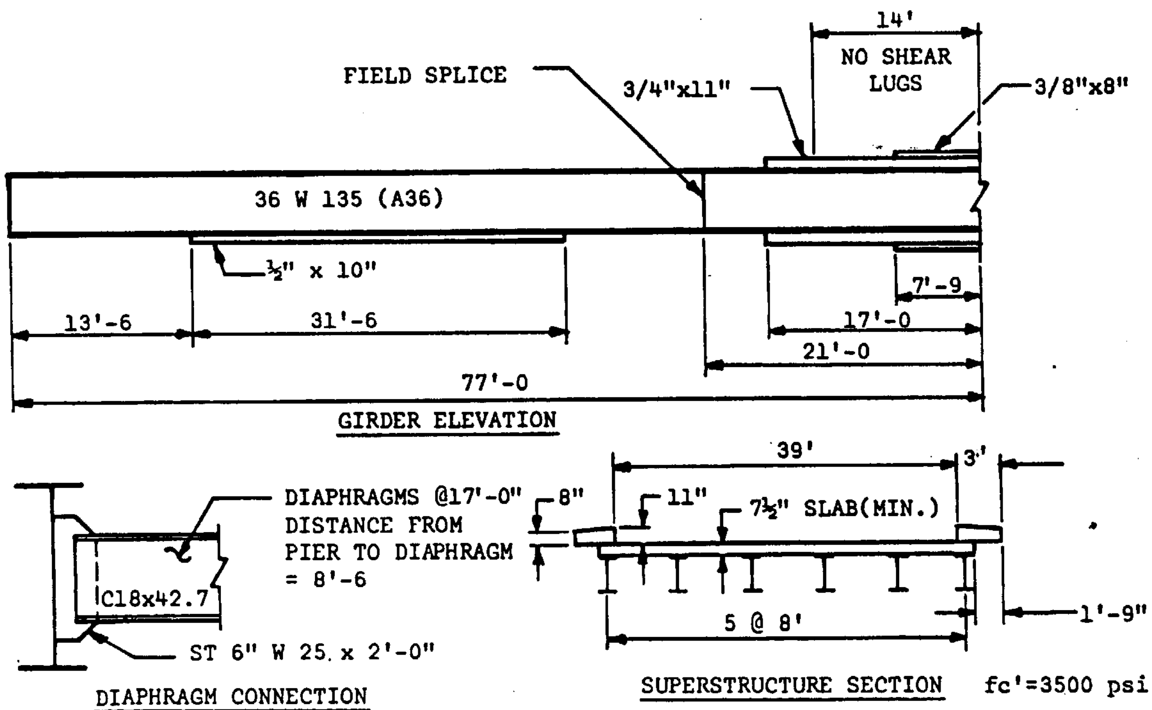
Summary

Inventory Rating …………………..HS17

Operating Rating ………………….HS29

(3) Two Span Composite Rolled Steel Girder (B-13-191)

The following bridge was built in 1961 and has no overlay or corrosion. The roadway width is 39' so three lanes must be applied. Only the interior girder is rated for this example.



The inventory rating for this girder is controlled by the allowable fatigue stress in the base metal at the end of the cover plates. This bridge is carrying a major highway and must be checked for a stress cycle of 500,000 for trucks. The allowable range of stress category "E" equals 13 ksi using AASHTO 1987 Interim Specifications Bridges.

Inventory Rating – Right End of ½” x 10” Cover Plate

Dead load moment = 197. ft. kips
Live load moment = 713. to –271. for HS20 truck
(Note: Live load moments are based on a distribution of S/5.5.)
Tension stress from dead load = 197(12)/438.6 = 5.39 ksi
Tension stress from live load 713(12)/599.2 = 14.28 ksi
Compression stress from live load and dead load (-271+197)(12)/438.6 = -2.02 ksi
Actual stress range equals 5.39+14.28+2.02 = 21.69 ksi

Because the use of two section modulii is required, the stress is not linearly related to the moment. The engineer must assume a rating and check it.

Try HS12
Tension stress from live load = 12/20(713)(12)/599.2 = 8.57 ksi
Stress from negative live load and dead load = (.6(-271.)+197.)12/438.6 = .94 ksi tension
Stress range equals 8.57-.94+5.39 = 13.02 ksi

Inventory Rating – End of ¾” x 11” Cover Plate

Dead load moment = -148. ft. kips
Live load moment = +391. to –352. for HS20 truck
Top Flange
Tension stress from dead load = 148(12)/438.6 = 4.05 ksi
Tension stress from live load = 352(12)/438.6 = 9.63 ksi
Compression stress from live load and dead load =(391-148)12/2996 = .97 ksi
Actual stress range equals 4.05+9.63+.97 = 14.65 ksi
Bottom Flange
Compression stress from dead load = 4.05 ksi
Compression stress from live load = 352(12)/438.6 = 9.63 ksi
Tension stress from live load and dead load = (391-148)(12)/599.2 = 4.87 ksi
Actual stress range equals 4.05+9.63+4.87 = 18.55 ksi
TRY HS14
Compression stress from live load = 14/20(9.63) = 6.74 ksi
Tension stress from live and dead load = (14/20(391)-148)(12)/599.2 = 2.52 ksi
Actual stress range equals 4.05+6.74+2.52 = 13.31 ksi
TRY HS13
Compression stress from live load = 13/20(9.63) = 6.26 ksi
Tension stress from live load and dead load = (13/20(391)-148)(12)/599.2 = 2.13 ksi
Actual stress range equals 4.05+6.26+2.13 = 12.44 ksi
SAY HS14.0

Inventory Rating End of 3/8" Cover Plate

Dead load moment –465
Live load moment + 112 to –365 for HS20 truck
Actual stress range equals 477(12)/714.3 = 8.01

The inventory rating for this bridge is HS12 and is controlled by fatigue at the end of the1/2" x 10" cover plate.

Operating Rating at Pier

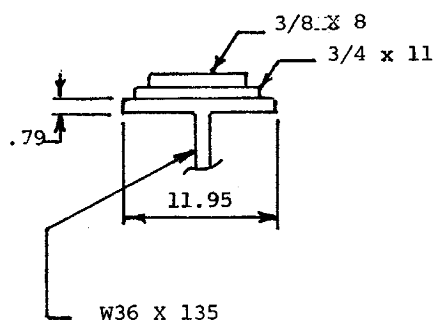
Check web thickness to see if section meets the compact section requirements.

$$d/t \leq 13330/\sqrt{f_y}$$

$$d/t = 33.97/.6 = 56.6$$

$$13300/\sqrt{36000} = 70.1 \quad OK$$

Check lateral bracing requirements.



Determine moment of inertia and radius of gyration with respect to the y-y axis.

$I_y = 225 + 1/12\ bh^3$
$I_y = 225 + 1/12*1.5*11^3+1/12*.75*8^3$
$I_y = 225+166. + 32 = 423.$
$A_y = 39.7+11*1.5+8*.75 = 62.2$
$r_y = \sqrt{I/A} = \sqrt{423/62.2} = 2.61$
$Lb/r_y = 8.5*12/2.61 = 39.1$

Assume HS40 rating
From computer run with HS40 live load
M_1 & M_2 are service loads at adjacent braced points.

$M_1 = 794 + 1200 = 1994$

$M_2 = 465 + 728 = 1193$
$.7*M_1 = 1396$

$M_2 < .7M_1$

When $M_2 < 0.7\ M_1$ Lb/r_y must be less than

or equal to $12000/\sqrt{F_y}$ to meet compact section requirements.

$12000/\sqrt{F_y} = 63$

Check maximum shear force

$V \leq .55\ f_y\ dtw$

.55*36*35.55* .6 = 422. kips

VDL + VLL = 46.5 + 120 = 166 OK

Pier section meets compact section requirements.

Determine plastic within modulus.

$Z = 509 + \Delta Z$

ΔZ equals increase in Z due to cover plates = $A_p\ (d+t_p)$

Z = 509+8.25*36.3+3*37.4 = 921. In^3

$M_u = F_y Z$ = 36*921/12 = 2763 ft. kips

M_u = 1.3*D + 1.3*RF*(L+I)

RF = (2763 – 1.3*794)/(1.3*602)

RF = 2.21

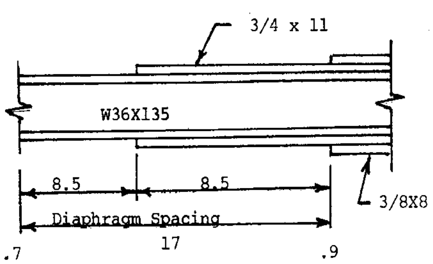
Check Serviceability Strength

$RF(L+I) < .8F_y\ S\text{-}D$

I = 7800+2*8.25*18.15^2 + 2*3*18.7^2
I = 7800+5435+2098 = 15333.
S = I/C = 15333/18.9 = 811.
RF*602 = .8*36*811./12 – 794
RF = 1.91

Serviceability Strength Controls

<u>Operating Rating at End of 3/8" Cover Plate</u>
Check lateral bracing requirements.



$I_y = 225 + 1/12\ bh^3$

$= 225 + 166 = 391$

$A = 39.7 + 16.5 = 56.2$

$r_y = \sqrt{I/A} = 2.65$

$Lb/r_y = 17*12/2.64 = 77.3$

Compact Section requirements not satisfied for lateral bracing.

Spacing of lateral bracing for a Braced Non-Compact Section.

$Lb \leq 20{,}000A_f / F_y d$

$\leq 20{,}000*13.56/(36*36)$

$\leq 209"$

12*17 = 204. OK when

A_f = average flange area.

Section is Braced Non-Compact

$M_u = F_y S$

$I = 7800 + 2*8.25*18.15^2 = 13235$
$S = I/c = 13235/18.52 = 714$
$M_u = 36*714/12 = 2142$ ft. kips

$M_u = 1.3*D + 1.3*RF*(L+I)$

RF = (2142 – 1.3*465)/(1.3*365)
RF = 1538/474 = 3.24

<u>Operating Rating at End of ¾" Cover Plate Based on Negative Moment</u>

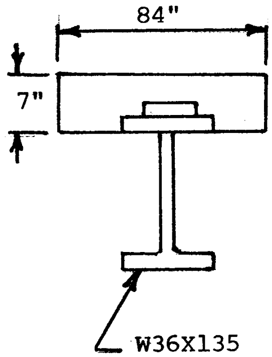
Section is Braced Non-Compact so

$M_u = F_y S$

$M_u = 36*439/12 = 1317$ ft. kips
$M_u = 1.3*D + 1.3*RF(L+I)$
RF = (1317 – 1.3*148)/ (1.3*352)
RF = 1125./458 = 2.46

<u>Operating Rating at Right End of ½" Cover Plate</u>

Composite action takes place at this location and for the steel section to qualify as compact only AASHTO Article 10.48.1.1(b) and 10.48.1.1(e) need be met. These conditions are met so the method described in AASHTO 10.50.1 can be used to compute the maximum moment strength. However, the method results in unrealistically high moment capacities and serviceability requirements will usually control. Wisconsin Bridge Office practice is to base moment strength for positive moment composite section on "Non-Compact Section" criteria.



$M_{L+I} = S^* \; [F_y - 1.3(D1/SD_1 - D2/SD_2)]$

Use n = 9 for operating rating

S = 604 = composite section modulus for bottom of girder from computer program.

$M_{L+I} = 604/12[36 - 1.3*197*12/439] = 1460$ ft. kips

$M_{L+I} = 1.3*RF*(L + I)$

$RF = 1460/(1.3*713)$

$RF = 1.58$

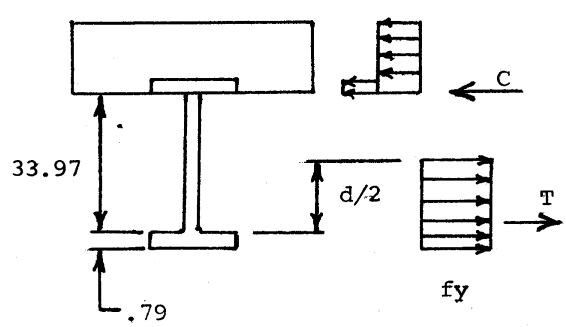
Check serviceability strength

$RF(L+I) < S^*[.95\, F_y - D1/SD_1 - D2/SD_2]$

where S equals composite section modulus based on n = 9, SD_1 equals section modulus of steel alone and D1 equals non-composite dead load and SD_2 equals composite section modulus based on n = 27 and D_2 equals composite dead load.

$RF*713 = 604/12[34.2 - 197*12/439]$
$RF = 604/713/12(34.2 - 5.38)$
$RF = 2.03$

Another method which is more conservative than AASHTO 10.50.1.1 is to assume the bottom flange and the lower half of the web will reach yield stress to develop a plastic moment. The resultant moment capacity is then determined by assuming the resultant of the compressive forces at the bottom of the slab. This method is illustrated below.



$T_f = 36*11.95*.79 = 340.$ kips

$T_w = 36*.6*17 = 367.$ kips

$M_u = (340*33.6+367*25.5)/12.$

$M_u = 1732.$ ft. kips

$M_u = 1.3*D+1.3*RF(L+I)$

$RF = (1732-1.3*197)/(1.3*713) = 1.59$

<u>Operating Rating at Point of Maximum Positive Moment (.4 Point)</u>

D = 340 ft. kips
LL+I = 832 ft. kips
S_b = 769 (Composite Section)
S_b = 563 (Non-composite Section)
$M_{L+I} = S*[F_y - 1.3(D1/SD_1)]$
M_{L+I} = 769/12[36 – 1.3*340*12/563] = 1703. ft. kips
M_{L+I} = 1.3*RF*(L+I)
RF = 1703/(1.3*832) = 1.57

Permit Vehicle Rating

The moments from a 250 kips permit vehicle using a distribution factor of S/7 are as follows:

At Pier	1049 ft. kips
At .4 Point	1365 ft. kips

Live load capacity at pier = RF*(L+I)
for HS20 = 1.91*602.4 = 1150 ft. kips

Live load capacity at .4 point = RF*(L+I)
for HS20 = 1.57*832 = 1306 ft. kips

Permit Vehicle Rating = 1306/1365*250 = 240 kips

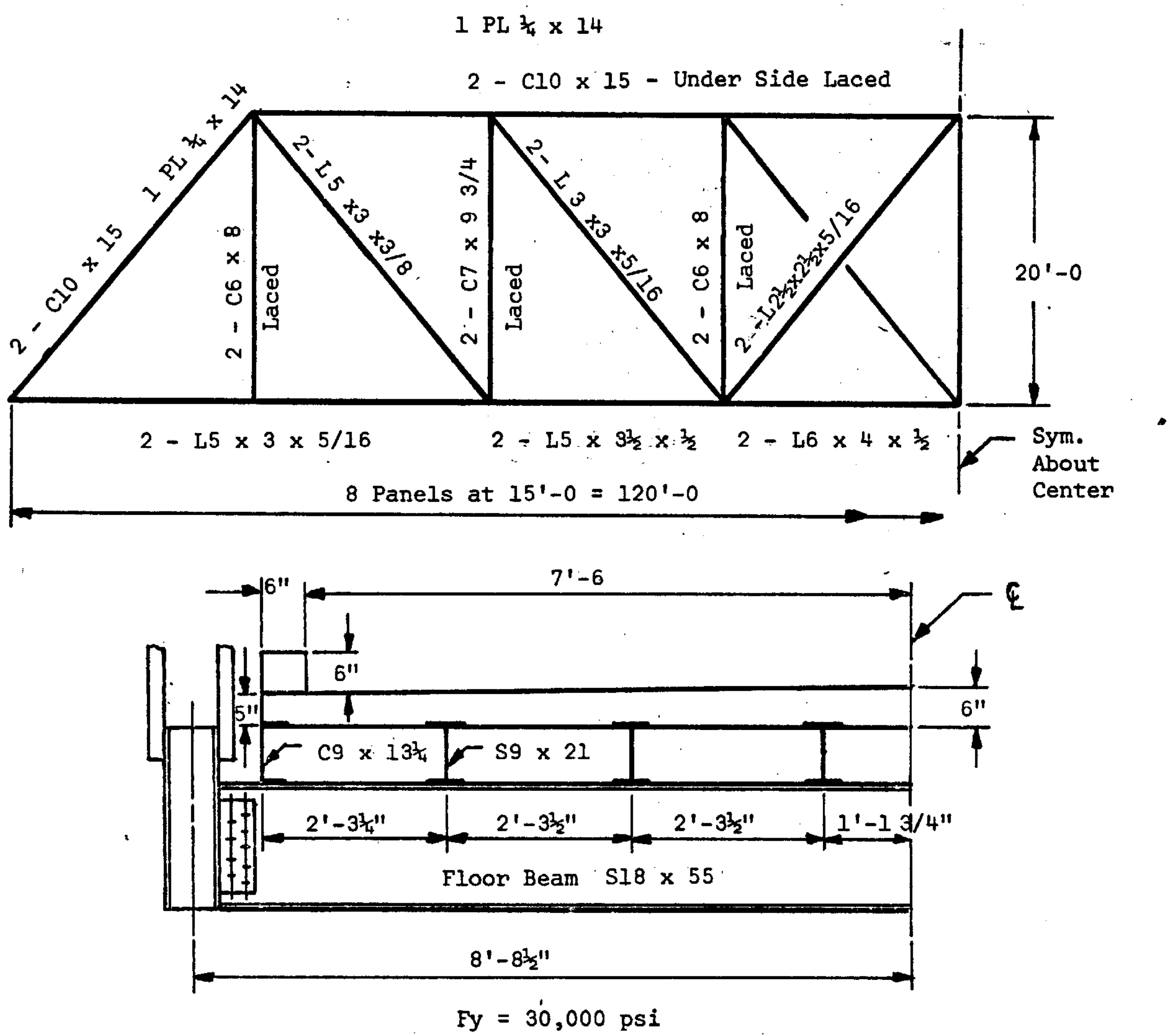
Summary

The inventory rating of this bridge is HS12 and is controlled by fatigue. The weak point is at the end of the ½" x 10" cover plate near the .6 point. Operating rating is controlled by the .4 point and equals 1.57*HS20 = HS31.

Inventory Rating …………HS12
Operating Rating ………..HS31
Permit Vehicle Rating …..240 kips

(4) Single Span Truss (B-23-939)

The following overhead steel truss was built in 1914. The section loss for the truss members and stringers is assumed to be 5%. No loss in section of the floorbeam has occurred. A one-inch bituminous wearing surface has been added to the bridge. Fy = 30,000 psi.



Stringer Rating

The stringers are rated first since the roadway width is 15'-0, only one lane is applied. The fraction of a wheel load to a stringer equals S/7 = 2.292/7 = .327.

H20 and HS20 live load moment = 1.3(120/2).327 = 25.5 ft. kips
Uniform dead load = .5(2.29)(.15)+.083(2.29)(.144)+0.21 = .220 ft. kips

Dead load moment = $1/8(.220)15^2$ = 6.19 ft. kips

The stringers do not have adequate lateral bracing to qualify for compact sections. Check to see if they qualify as braced non-compact sections.

(a) Projecting compression flange element

$$b/t \leq 2200/\sqrt{F_y}$$
$$2.25/.44 \leq 12.7$$
$$5.1 \leq 12.7 \ OK$$

(b) Web thickness

$$D/tw \leq 150$$
7.8/.29 = 27 OK

(c) Spacing of lateral bracing

$$L_b \leq 20{,}000\, A_f/(F_y * d)$$
$$L_b \leq 20{,}000 * .44 * 4.5\ /(30 * 9)$$
$$L_b \leq 147$$

Assuming point of DL+LL contraflecture occurs near the middle of the span, the actual L_b = 7.5*12 = 90. Section qualifies as braced non-compact.

$M_u = F_y S$
M_u = 30.*.95*18.7/12 = 44.4 ft. kips

Assuming 5% section loss from corrosion.

For Operating Rating

M_u = 1.3 DL + 1.3*RF*(LL+I)

RF = (44.4-1.3*6.19)/(1.3*25.5)

RF = 1.10

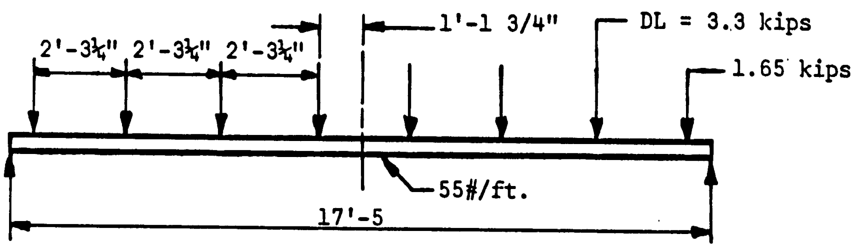
Operating Rating = 1.10*HS20 = HS22

Inventory Rating = .6* Operating Rating

Inventory Rating = HS13

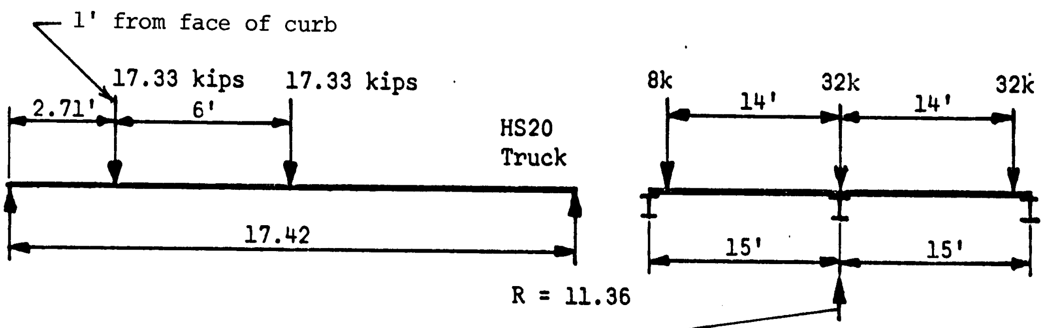
Floorbeam Rating

The floorbeam is rated next. The floorbeam length is the distance between centerlines of trusses. The dead load stringer reactions are shown below.



Dead Loads

R = 11.55 + .48 = 12.03 kips



Live Load

R = 32 + 1/15(8+32)
R = 34.67/axle

HS20 live load moment = 11.36(8.71)(1.3) = 128.6 ft. kips
Dead load moment = 12.03(8.71)-(9.9)(3.42)-(1.65)(7.96)
- .055(8.71)(8.71)/2 = 104.78 - 33.858 - 13.134 - 2.068 = 55.7 ft.kips

Check to see if floorbeam qualifies as compact section.

(a) Projecting compression flange element

$b'/t \leq 1600/\sqrt{F_y}$

$2.75/.46 = 5.97 \leq 9.24$ OK

(b) Web thickness

$d/t_w \leq 13{,}300/\sqrt{F_y}$

$18/.46 = 39 < 77$ OK

(c) Spacing of lateral bracing

L_b = distance between points of bracing of compression flange equals stringer spacing equals 27"

$r_y = \sqrt{I_y/A} = \sqrt{21/16.2} = 1.14$

$L_b/r_y = 27/1.14 = 24$

$L_b/r_y < 7000/\sqrt{F_y} < 40$ OK

Section can be considered a compact section.

$M_u = F_y Z$

For a wide flange section Z can be computed with reasonable accuracy from the equation.

$Z = (A-wd)*(d-t)/2.+wd^2/4$

A = beam area

w = web thickness

d = depth of beam

t = average flange thickness

$Z = (15.93 - .46*18)*(18-.69)/2.+.46*18^2/4$

$Z = 7.65*8.65 + 37.26 = 103.4$

Note: The shape factor, Z/S, for wide flange shapes varies from 1.10 to 1.18, the average being 1.14.

$M_u = 30*103.4/12 = 258.5$ ft. kips

Reduce 5% because of section loss

$M_u = .95*258.5 = 246.$ ft. kips

Operating Rating

M_u = 1.3DL+1.3RF*(LL+I)

RF = (246-1.3*55.7)/(1.3*128.6)

RF = 1.04

Operating Rating = 1.04*HS20 = HS21
Inventory Rating equals .6* Operating rating = HS13

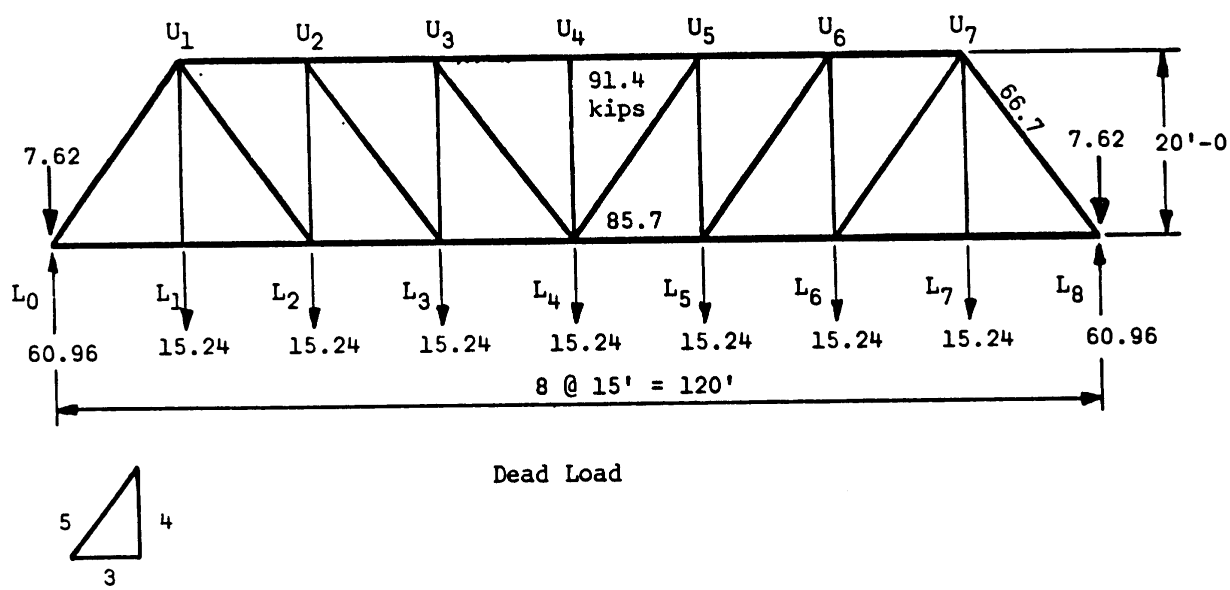
The capacity of the rivets connecting the floor beam to the truss must also be checked.

There are 10 – ¾" rivets in single shear
The allowable operating stress is 18 ksi
Working capacity = 10*.441*18 = 79 kips

DL + LL+I < 79 OK

Truss Rating

The capacity of the truss must now be calculated. The forces at each panel point from dead load must be calculated or obtained from the original calculations. The total steel weight for this bridge is 81,460 pounds. 81460(.5)/8 panels = 5091# steel/panel. The concrete and overlay equals 15(.458)8(.15) + .25(.15) + .083(7.5).144 = 10.15 kips. Dead load/panel = 15.24 kips/panel.



Dead Load

For live load the number of wheel lines per truss = 14.708/17.42 + 8.708/17.42 = 1.344. The wheel load is placed 1.5' from the face of curb. The maximum LL moment at any point on the span can be obtained by using formulas given in AASHTO Manual for Maintenance Inspection of Bridges.

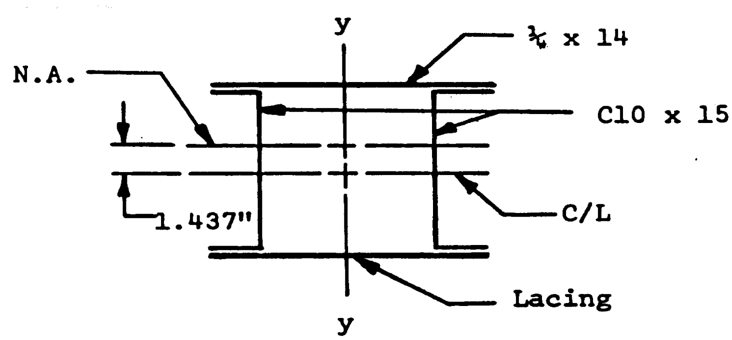
For HS20 loading moment per wheel line equals:

M = 36(L-X) (x-4.67)/L – 56
M = 36*.5*(60-4.67)-56 = 940 ft. kips
M design = M*DF = M*1.344 = 1263.

M_d = (U3 – U4)*20

U3-U4 = 63.2 kips without impact
I = 50/(L+125) = .2
U3-U4(LL+I) = 63.2*1.2 = 75.8 kips

Section properties of U3-U4 must be determined before the allowable stress can be calculated.



Area = 2(4.49) + 3.5 = 12.48 inches^2
y = 3.5(5.125)/12.48 = 1.437

I = 3.5(3.688)^2 + 2(67.4) + 2(4.49) (1.437)^2

I = 47.60 + 134.8 + 18.54 = 200.9 inches^4

r^2 = I/A = 200.9/12.48 = 16.10
r = 4.01 about N.A.
Iyy = 1/12(.25)(14)^3 + 2(4.49)(5.034)^2

+2(2.28) = 289.2 inches^4
1/r = 15(12)/4.01 = 44.89
C_c = 23,926 F_y = 23,926/173.2 = 138.1

From Appendix C in AASHTO Specifications the value of K equals .75, so K L/r = 33.67.

$Fa = F_y/FS(1-((KL/r)^2 F_y/4^2 E))$

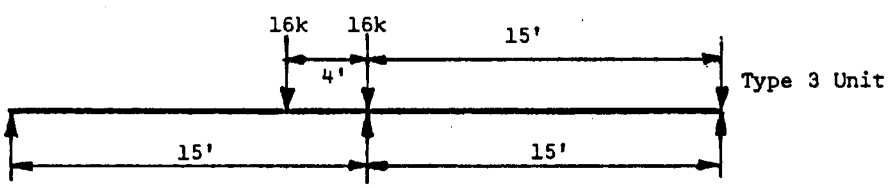
Fa = 30,000/FS(1-.030) = 29,100

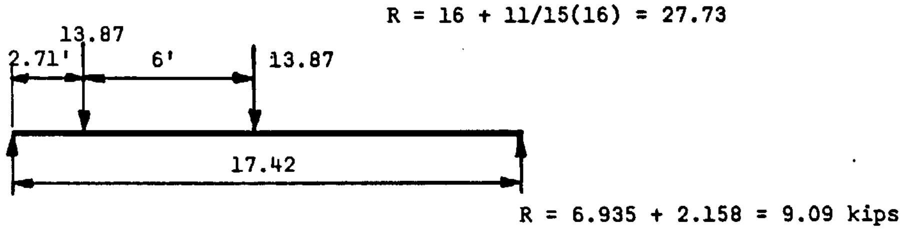
Where Fa is the allowable compression stress and FS is the factor of safety. This is reduced 5% due to section loss. FA = 27,645/FS

For operating rating Fa = 27,645/1.7 = 16,262 psi
Dead load stress = 91.4/(.95(12.48)) = 7.71 ksi
Live load stress = 75.8/(.95(12.48) = 6.39 ksi
Operating rating (U3-U4) = 16.26-7.71/6.39(20) = HS27

For purposes of illustration, assume member U3-U4 is the most overstressed member and therefore the controlling member. Then the overall capacity of this truss is governed by the floorbeam. Since the operating rating is low the capacity using Type 3 and Type 3S2 vehicles is also determined.

<u>Floorbeam Rating for Statutory Loads</u>





Type 3 unit and Type 3S2 unit live load moment = 9.09(8.71)(1.3) = 102.9 ft. kips

M_u = 246 ft. kips

M_u = 1.3DL + 1.3 RF*(LL+I)

RF = (246-1.3*55.7)/(1.3*102.9) = 1.30
Type 3 unit maximum load = 1.3*23 tons = 30 tons > 23 OK
Type 352 maximum load = 1.3*36 tons = 47 tons > 36 OK

Posting

The floorbeam capacity governs. This bridge can carry statutory loads but restrictions must be placed on the "Annual Permit Loads". Limit gross vehicle weight to 45 tons and 3 axle vehicles to 30 tons.

Summary

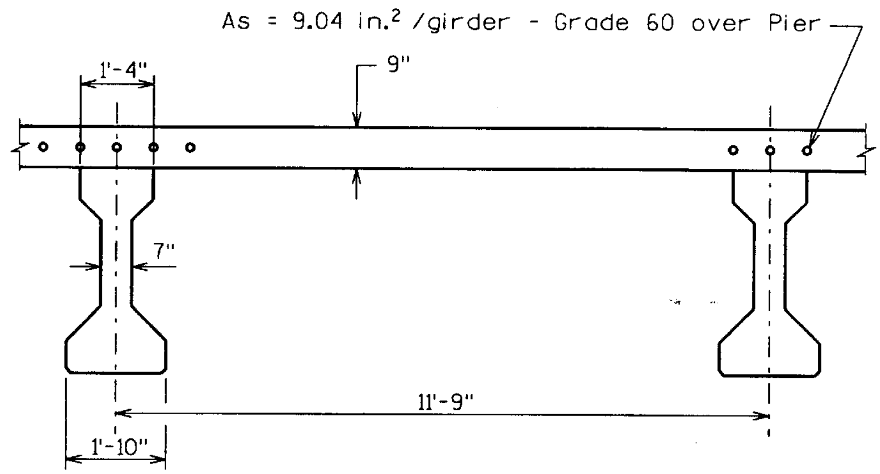
Inventory Rating HS13
Operating Rating HS21

Advisory Posting

3 Axle Vehicles - 30 Tons
Gross Weight - 45 Tons

(5) Two-Span Prestressed 45" Girder

Designed in 1975 using 6000 psi concrete, the girder is 45" deep with 28 /12 Inch stress relieved strands. Only an interior girder will be rated. Assume the girder is embedded ½" into the 3500 psi concrete slab.



Span Lengths – 62'-0, 62'-0

Girder Properties

$A = 560$ sq. in.
$I = 125,390$
$S_B = 6186$
$S_T = 5070$
$Y_B = 20.27"$
$Y_T = 24.73$
$E_s = 16.41"$
Modular Ratio $N = E_s/E_c = 5$.

Composite Section Properties

$I = 372,716$
$S_B = 10,479$
$S_T = 39,522$
$S_B = 35.57"$
$S_T = 9.43"$
Equivalent Area of Slab Factor = 0.7

AASHTO specifications have formulas that give the theoretical moment strength of a prestressed section. The girders have enough web reinforcement extended into the slab to develop full composite action at ultimate moment.

Positive Moment Capacity – Load Factor Method

For prestressed girders with composite slabs the theoretical moment capacity may be assumed as:

$Mt = A^*_s (f^*_{su})\ d (1-0.6\ p^*(f^*_{su}) /f'c)$
$f^*_{su} = f's (1-0.5\ p^*\ f's/f'c)$
A^*_s = area of prestressing steel = 28 x 0.1531 = 4.287"
d = distance from extreme compressive fiber in slab (deduct 1/2" for wearing surface) to centroid of the prestressing strands = 45 + (9-0.5-0.5)-3.86 = 49.14"
b - effective width of composite slab = 8.5 x 12 + 7 = 109"
$p^* = A^*_s/bd$, ratio of prestressing steel = 4.287/(109 x 49.14) = .00080
f'c = compressive strength of concrete slab
f's = ultimate strength of prestressing steel
f^*_{su} = 270 [1-0.5(.0008)(270/3.5)] = 262 Ksi
Mt = 4.287 x (262)49.14(1-0.6(.0008)262/3.5)/12 = 4434 ft. kips

This is the theoretical moment strength and it must be reduced by the capacity reduction factor since the slab is non-factory produced concrete. M_u = .95Mt = .95(4434) = 4213 ft. kips.

Dead Load Moment

The dead load of girder and slab = .583 + .75(11.75)(.15) = 1.905 kips/ft.

Dead load moment = $1/8(1.905)(62)^2$ = 915 ft. kips. There is a concrete diaphragm at C/L of span which weighs 3.4 kips. DLM =PL/4 = 3.4 x 62/4 = 52.7 ft. kips

Total dead load moment = 967.7 ft. kips.

Live Load Moment

The live load moment from an HS20 truck using a distribution factor of S/5.5 equals 893.7 at the .5 point of span.

Operating Rating

Operating rating = (.75(4213)-967.7)/(894)20 = HS49.

Although AASHTO specifications currently specify using 75% of the ultimate moment capacity, it is more consistent with load factor rating to use load factors of 1.3 for dead and live load. Using this approach the factored dead load equals 1.3(967.7) = 1258 ft. kips.

Operating rating = HS20(Mu - 1.3 M_{DL})(1.3 x M_{LL}) = (4213-1258)[1.3(894)]20 = HS51

Operating Rating Based on Stress of 90% of Yield

The requirement limiting the stress to 90% of yield in the bottom tendons must also be checked.
The stress in the strands without any live load equals the original stress of .7f's - 45,000 psi losses (assumed) plus the increase in stress due to slab and diaphragm dead load.

(Use .75f's for Low Relaxation Strands).

$f_s = .7(f'_s)\text{-}45+M_{DL}(Y_B\text{-}2)N/I_G$
$f_s = .7(270)\text{-}45 + 967.7(12)18.27(5)/125390 = 144 + 8 = 152$ ksi

The concrete stress in the bottom of the girder after all losses equals $f_c = -\frac{Pe}{S_B} - \frac{P}{A} + \frac{M_{DL}}{S_B} =$

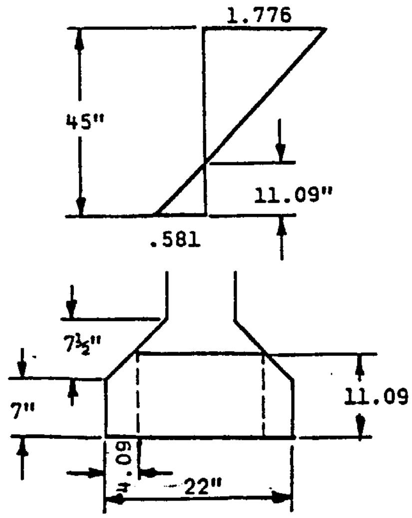
-144(28) (.1531)(16.41)/6186-144(28)(.1531)/560+(967.7)12/6186 = -.862 ksi where a negative sign means compression. The live load moment required to change the girder stress from -.862 ksi to the modulus of rupture $(7.5\sqrt{6000})$ $equals\, \Delta f_c(S_B)/12.$

$M_{LL} = (.862+.581)(10479)/12 = 1260\, ft.kips$

The bottom strand stress just before cracking equals f_s + (M_{LL} x (Y_B - 2))/I) x N = 152 + 1260(12)5(33.57)/372716 = 159. The top of girder concrete stress with a live load of 1260 ft. kips =

$$\frac{Pe}{S_T} + \frac{P}{A} - f_{DL} - f_{LL} = 617(16.41)/5070 - 1.102 - 967.7(12)/5070 - 1260(12)/39522 = -1.776.$$

where P = (.7(270)-45)4.287 = 617

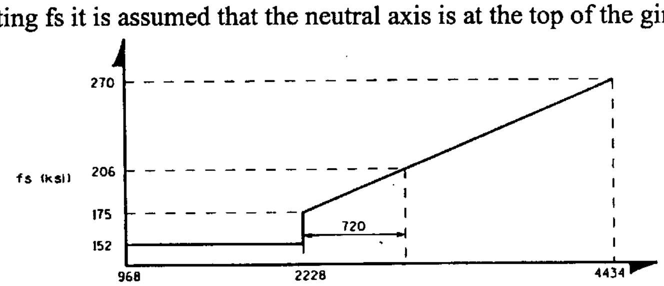


The force in concrete in tension equals 11.09(13.82)(.5)(.581) + 8.18 (9)(.581)(.5) = 65.93 kips

When the force in the concrete is transferred to the steel, the increase in steel stress for the strands nearest the bottom equals: F/As • (cg. F/c.g. AS = Δfs_{cg} = 65.93(45-11.09/3)/4.287(16.41 + 24.73) = 15.44

where $e_s + y_t$ = 16.41 + 24.73 = 41.14 and A_s = 4.287
Δ fs = 15.44(45-2)/41.14 = 16.14 ksi

When calculating fs it is assumed that the neutral axis is at the top of the girder.



The strand stress just after cracking equals 159 + 16 = 175 ksi. The allowable is .9 of yield = .9(229) = 206 ksi. The stress will now increase linearly from 175 to f's as the moment increases from 2228 ft. kips to 4434 ft. kips, 4434 being the theoretical moment capacity and 2228 (DLM+LLM = 968 + 1260) is the total moment when the strand stress equals 175.

M = (4434-2228)(206-175)/(270-175) = 2206*31/95 = 720

The live load capacity equals 1260 + 720 equals 1980 ft. kips.

Operating rating based on an allowable stress of 90% of yield in the bottom tendons equals (1980/894)20 = HS44.

<u>Operating Rating Based on Continuity Steel over Pier - Load Factor Method</u>

The operating rating based on negative moment over the pier must also be determined. Tests have shown that even though the concrete between the girders is of lower strength than the girder concrete, the theoretical moment capacity is more accurately determined by using f'_c for the girder.

f'_c = 6000 and fy = 60000

Ru = p fy(1-0.5 p m)
m = fy/(0.85fc') = 60/5.1 = 11.76
p = 9.04/[22(49.5)] = .0083
Ru = .0083(60000)(1-0.5(.0083)11.76) = 473.6

Mt = Ru bd^2 = 473.6(22)49.5^2/12,000 = 2127 ft. kips

A check is made to see if neutral axis is within the bottom flange.

a(.85)(6)(22) = 9.04(60)
a = 4.83 O.K.

For factory produced concrete a capacity reduction factor of 1 may be used. However, since the concrete between the girders is field poured, a value of .95 for flexure seems reasonable. 706.6 is the moment over the pier from an HS20 lane load using a distribution factor of S/5.5.

Mu = .95(2127) = 2020 ft. kips
Operating rating = (Mu/1.3 M_{LL}) HS20 = 2020/[706.6(1.3)] 20 = HS43.9

<u>Inventory Rating</u>

The inventory rating in the positive moment region is based on the allowable design stresses using working loads. There is no load factor Inventory Rating. The allowable design stresses equal $6\sqrt{6000}$ in tension and .4(6000) or .6(6000) in compression depending on the loading combination being checked. From calculation under "Operating rating based on stress of 90% of yield", the concrete stress in the bottom of the girder after losses of 45,000 psi equals - .862 ksi (compression). The live load moment required to change the girder stress from -.862 ksi to $6\sqrt{6000} = (.862 + .465)(10479)/12 = 1159\,ft.kips.$

Inventory rating = (1159/894)20 = <u>HS25.9</u> based on allowable tension in bottom of girder. Allowable compression does not control for this girder. Inventory rating in the negative moment region based on load factor design.

Inventory Rating = HS20 x (Mu/(2.17 x M_{LL}) = 2020(706.6)(1.3)5/3)20 = HS26.4

<u>Inventory Rating in the Negative Moment Region Based on Fatigue (Working Stress Method)</u>

Using ACI design tables:

Allowable fatigue stress = 23.4 ksi

m = NAs/bd = 10(9.04)/ [22(47.5)] = .0830

k = .333, j = 1-1/3k = .889

f_s = 12,000M/jdAs)

M = 23400 (.89(49.5)9.04)/12000 = 776.6 ft. kips

Inventory Rating (fatigue) = (776.6/706.6)20 = HS22.0

<u>Standard Permit Vehicle Rating</u>

From a computer run or design aids the moments per wheel line of a 250 kip permit vehicle including impact are as follows:

Point	Moment (ft-kips)
.4	844
.5	821
1.0	-872

The distribution factor for single lane distribution equals S/7.0 = 11.75/7 = 1.68.

Using the .5 span point the moment from a 250 kips permit vehicle equals 1.68*821 = 1379 ft. kips. From previous calculations for "Operating Rating Based on Stress of 90% of

Yield", the live load moment capacity equals 1980 ft. kips.

Max. Positive Permit Vehicle Capacity = (M_{CAP}/M_{LL}) x 250 = (1980/1379) x 250 = 359 kips at .5 point of span

Over the pier M_u = 2020 ft. kips, there is no dead load stress on the bar steel in the slab so:
Max. Neg. Permit Vehicle Capacity = (M_{CAP}/M_{LL}) x 250 = 2020/(1.3*872*1.68) x 250 = 265 kips

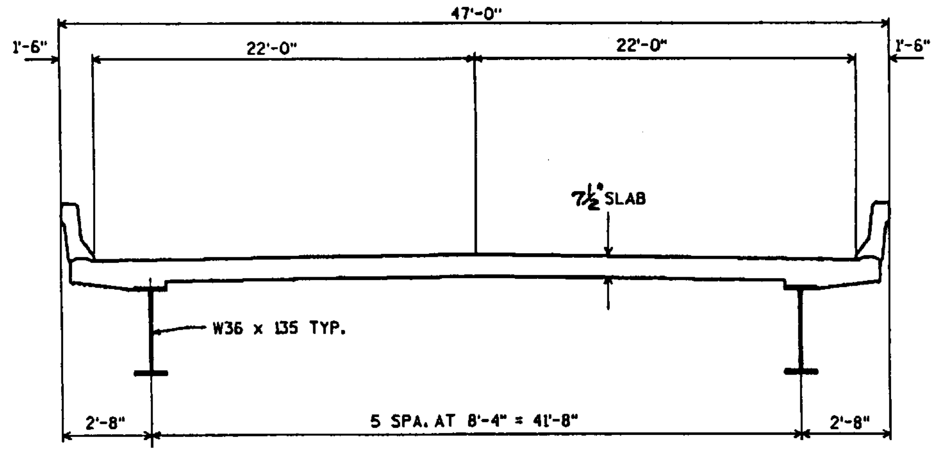
The maximum permit vehicle reported on plans or in the Bridge File is 250 kips.

Summary

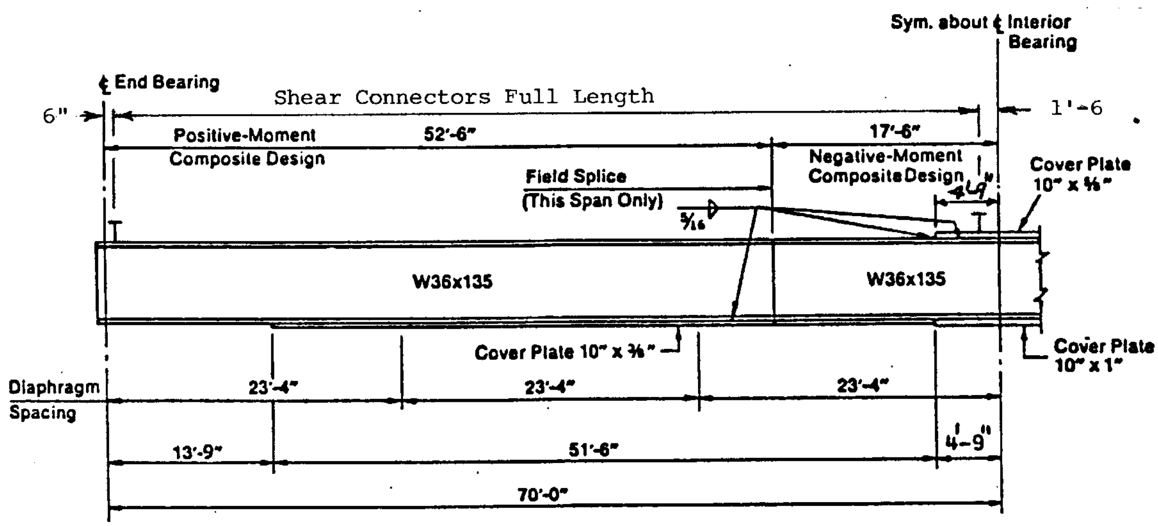
Inventory Rating ------------------------- HS22
Operating Rating ------------------------- HS44

Standard Permit Vehicle Rating ------- 250

(6) Two Span Rolled Steel Girder with Negative Moment Composite Action
Use fatigue cycles of 500,000. f_c' = 4000 psi.

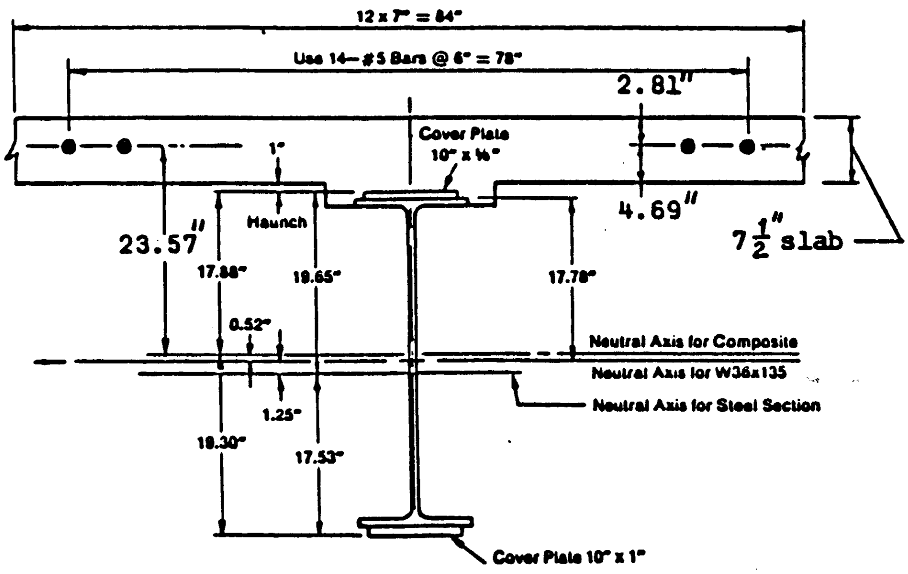


TYPICAL SECTION



BEAM ELEVATION

Note: This detail with cover plates is currently not used by the Bridge Office. Existing steel bridges with cover plates require fatigue check. In the negative moment regions, shear connectors must be provided when the reinforcing steel embedded in the concrete is considered a part of the composite section.



NEGATIVE-MOMENT SECTION

RATE SECTION AT INTERIOR SUPPORT

Rolled Section W36 x 135

Section Properties (Steel Construction Manual, Seventh Edition)

t_f = 0.794 inch	F_y = 36,000 psi	Area of Girder = 39.8 sq. in.
b_f = 11.945 inch	S_x = 440 in^3	Bottom Plate = 10” x 1”
t_w = 0.598 inch	Z_x = 510 in^3	Top Plate = 10” x 5/8”
d = 35.55 inch	r_y = 2.39	r_y with Cover Plates = 2.54 in.

Design Data:

Total Dead Load per Girder DL1 = 900 #/Ft.

Composite Dead Load (Parapets, Etc.) per Girder DL2 = 165 #/Ft.

Structure is designed for HS20 live loading.

L.L. Distribution Factor = 8.33/5.5 = 1.51

Impact = 50/(70+125) = 0.256

MDL1 (Slab DL + girder) Moment = 551 kip-ft

MDL2 (Parapets, etc. DL) Moment = 101 kip-ft

M(L+I) (Live Load and Impact) Moment = 601 kip-ft

VDL1 (Slab and Girder DL) Shear = 39.3 kips

VDL2 (Parapets, etc. DL) Shear = 7.2 kips

V(L+I) (Live Load and Impact) Shear = 62.1 kips

f'_c (Slab) = 4,000 psi

Grade 40 Reinforcing Steel F_y = 40,000 psi

Over the interior support, the slab reinforcement steel is used as part of the section.

Section Properties

Steel Section at Interior Support

Material	A	d	Ad	Ad^2	I	I
W36x135	39.80				7,796	7,796
Bottom Plate 10x1	10.00	-18.28	-182.8	3,342		3,342
Top Plate 10x5/8	6.25	18.09	113.1	2,045		2,045
	56.05 in^2		-69.7 in^3			13,183
			-1.25x69.7 =			-87
			I (Neutral Axis)		=	13,096 in.^{4}

$$d = \frac{-69.7}{56.5} = -1.25 \text{ in.}$$

d(top of steel) = 17.78 + 0.62 + 1.25 = 19.65 in.

d(bot. of steel) = 17.78 + 1.00 - 1.25 = 17.53 in.

$$S(\text{top of steel}) = \frac{13{,}096}{19.65} = 666 \text{ in.}^3$$

$$S(\text{bot. of steel}) = \frac{13{,}096}{17.53} = 747 \text{ in.}^3$$

Composite Section at Interior Support:

Material	A	d	Ad	Ad^2	I	I
Steel Section	56.05	-1.25	-69.7			13,183
Reinf. Steel 14 No. 5	4.34	23.34	101.3	2,364	-	2,364
	60.39 in.^{2}		31.6 in.^{3}			15,547
				-0.52x31.6 =		- 16
			I (Neutral Axis)			= 15,531 in^4

$d_r = \dfrac{31.6}{60.39} = 0.52$ in.

d(reinforcement) = 17.88 + 1.00 + 4.69 = 23.57"

S(reinforcement) = $\dfrac{15{,}531}{23.57} = 659$ in.^{3}

d(top of steel) = 17.78 + 0.62 – 0.52 = 17.88 in.

d(bot. of steel) = 17.78 + 1.00 + 0.52 = 19.30 in.

S(top of steel) = $\dfrac{15{,}531}{17.88} = 869$ in.^{3}

S(bot. of steel) = $\dfrac{15{,}531}{19.30} = 805$ in.^{3}

Check Local Buckling Requirements for Flange:

Current AASHTO Specifications followed to check all requirements.

When checking flange with cover plate, use the maximum width of the flange or the cover plate and the total thickness of both beam flange and cover plate.

b_{max} = 11.945 inch t = 0.794 + 1.0 = 1.794 inch

b' = $\dfrac{11.945 - 0.598}{2} = 5.674$ inch

$b'/_t = \dfrac{5.674}{1.794} = 3.16$

$$\frac{1600}{\sqrt{Fy}} = \frac{1600}{\sqrt{36{,}000}} = 8.43 \qquad \text{so } b'/_t \langle \frac{1600}{\sqrt{Fy}}$$

Check projection beyond cover plate weld.

b' = $\frac{11.945 - 10.00}{2} = 0.972$

t_f = 0.794" b'/t_f = $\frac{0.972}{0.794} = 1.22 \langle$ 8.43

Check Web for Buckling

$d/_{tw} \leq \frac{13{,}300}{\sqrt{F_y}}$ where d = Depth of basic section

$d/_{tw} = \frac{35.55}{0.598} = 59.45 \langle \frac{13{,}300}{\sqrt{36{,}000}} = 70.1$

Web will not buckle

Check Compression Flange Bracing Requirements

Find unbraced length

Bracing length = 23'-4"

M_2 < 0.7 M_1

M_2 = 314 < 0.7 x 1253 = 877

O.K. for Lateral Bracing

Use $L_b/r_y \leq \frac{12000}{\sqrt{Fy}}$

Check Unbraced Length

$L_b/r_y \leq \frac{12000}{\sqrt{F_y}}$

$\frac{23.3x12}{2.54} = 110.$ $\frac{12000}{\sqrt{36000}} = 63.2$

so $L_b/r_y > \frac{12000}{\sqrt{F_y}}$

Section is not adequately braced for a compact section.

A compact section is one in which the geometry is such that the section can be brought to full plastic moment without local buckling, and for which the bracing of the compression flange is adequate to prevent lateral torsional buckling. The cross-sectional geometry is defined by b'/t and d/t_w ratios which are not allowed to exceed certain AASHTO recommended values shown above if the section is to qualify as compact.

Check if section is a braced non-compact section.

$$L_b \leq \frac{20{,}000{,}000}{F_y \, x \, d} \quad x \; A_f$$

$$L_b = 23.3 \; x \; 12 = 280" \langle \; \frac{20{,}000{,}000 \, x \, (11.945 \, x \, 0.794 + 10.0 \; x \; 1.0)}{36{,}000 \; (35.55 + 1.0 + 0.625)} \qquad \langle \; 291.2$$

Bracing is adequate for a braced non-compact section and the section meets all of the requirements of a braced non-compact section. The compactness is controlled by the diaphragm spacing.

Since the section does not meet the requirements of a compact section at the support, the moments cannot be redistributed from the support to the spans.

Find Moment Capacity

$M_u = F_y$ x S controlling equation.

Available Moment in Top Flange

$$M_t = S(L+I) \; x \; (F_y - \frac{1.3 \, x \, MDL1}{SDL1} - \frac{1.3 \, x \, MDL2}{SDL2})$$

$$M_t = 869 \; x \; (36.00 - \frac{1.3 \, x \, 551 \, x \, 12}{666} - \frac{1.3 \, x \, 101 \, x \, 12}{869}$$

$$M_t = 18493 \, kip - inch = 1541 \, kip - ft.$$

Available Moment in Bottom Flange

$$M_b = 805 \; x \; (\; 36.00 \; - \frac{1.3 \, x \, 551 \, x \, 12}{747} - \frac{1.3 \, x \, 101 \, x \, 12}{805} \;)$$

$$M_b = 18141 \; kip - inch = 1512 \; kip - ft \;\; Controls$$

Find Moment Rating Factor (Operating Rating)

$$\text{MRF} = \frac{M(Control)}{1.3xM(L+I)} = \frac{1512}{1.3\,x\,601} = 1.935$$

Check Reinforcement Stress

$$F_y \geq \frac{1.3\;(MDL2 + MRF\;x\,(M\,(L+I)))}{S_{re\,\mathrm{inf}}}$$

MDL1 is not loaded on the reinforcement steel.

$$F_y = 40.00 \rangle \frac{1.3\,x\,((101x12) + (1.935\,x\,601\,x12))}{659} \rangle\; 29.92\;ksi$$

Reinforcement steel does not control.

Check Shear Capacity at Interior Support

Live load shears are increased by the moment rating factor since moments and shears must be compatible for a given loading or rating.

V = 1.3 (VDL1 + VDL2 + MRF x V(L+I))

= 1.3 (39.3 + 7.2 + 1.935 x (62.1))

= 216.7 kips

Check shear capacity

$V_u \leq$ (1.27 x 10^8 x t_w^3) / D but not more than 0.58 F_yDt_w

$$V_u = 216.7 < \frac{1.27\,x10^8\;x\,(0.598)^3}{(35.55 - 2(0.794))}\;x\;\frac{1}{1000} \qquad 800 \text{ kips}$$

or

V_u = 216.7 $\leq$ 0.58 F_yDt_w

< 0.58 x (36.0) x ((35.55) – 2(0.794)) x (0.598)

< 424.1 kips Controls

This section is adequate for shear. If the shear was larger than the allowable, a rating factor based on the shear capacity would have to be used.

Find Serviceability Strength Rating Factor

$$SRF = \frac{S(L+I)}{M(L+I)}[0.95F_y - \frac{MDL1}{SDL1} - \frac{MDL2}{SDL2}]$$

Check bottom flange

$$= \frac{805}{(601)(12)} \; [0.95\;(36) - \frac{551x12}{747} - \frac{101x12}{805}\;]$$

= 2.661 Does not control

By observation, the reinforcement does not control serviceability strength. The controlling rating factor at this section, which is known as the Operating Rating Factor, is 1.935. Operating Rating is the maximum live load that may pass over a structure on an occasional basis. An Inventory Rating is the maximum live load which can safely utilize a structure for an indefinite period of time without damage. This rating can be taken as 0.6 times (reciprocal of 5/3) the Operating Rating.

Operating Rating = 1.935 x 20 = HS38

Inventory Rating = 1.935 x 0.6 x 20 = HS23

RATE SECTION 28 FEET FROM END (0.4 POINT OF SPAN)

MDL1 = 309 kip-ft Area of Girder = 39.8 sq. in.

MDL2 = 57 kip-ft Effective Width of Slab = 12(7.5”-0.5”) = 84” controls

M(L+I)= 750 kip-ft Effective Thickness of Slab = (7.5”-0.5”) = 7”

VDL1 = -1.5 kips

VDL2 = -0.3 kips

V(L+I) = 26.7 kips

Section Properties

Steel Section, 28 Feet from End Support:

Material	A	d	Ad	Ad^2	I_o	I
W36x135	39.80				7,796	7,796
Bot. Cover Plate 10x3/8	3.75	-17.96	-67.35	1,210		1,210
	43.55 in^2		-67.35 in^3			9,006
			-1.55x67.35 =			- 104
			I(Neutral Axis) =			8,902 $in.^4$

$$d_s = \frac{-67.35}{43.55} = -1.55\ in.$$

d(top of steel) = 17.78+1.55 = 19.33 in.
d(bot. of steel) = 17.78+0.38-1.55 = 16.61 in.

$$S(\text{top of steel}) = \frac{8{,}902}{19.33} = 460\ in.^3$$

$$S(\text{bot. of steel}) = \frac{8{,}902}{16.61} = 536\ in.^3$$

Composite Section, 3n = 24, 28 Ft. from end Support

Material	A	d	Ad	Ad^2	I_0	I
Steel Section	43.55	-1.55	-67.35			9,006
Conc. (84x7)/24	24.50	22.91	561.30	12,859	100	12,959
	68.05 $in.^2$		493.95 $in.^3$			21,965

$$d_{24} = \frac{493.95}{68.05} = 7.26\,in.$$

-7.26x493.95 = - 3,586

I(Neutral Axis) = 18,378 $in.^4$

d(top of steel) = 17.78 – 7.26 = 10.52 in.

d(bot. of steel) = 17.78 + 0.375 + 7.26 = 25.42 in.

$$S(\text{top of steel}) = \frac{18{,}378}{10.52} = 1{,}747\ in.^3$$

$$S(\text{bot. of steel}) = \frac{18{,}378}{25.42} = 723\ in.^3$$

Composite Section, n = 8, 28 Ft. from End Support

Material	A	d	Ad	Ad^2	I	I
Steel Section	43.55	-1.55	-67.35			9,006
Conc. (84.7)/8	73.50	22.91	1,684.00	38,578	300	38,878
	117.05 in.^{3}		1,616.65 in.^{3}			47,884

$d_8 = \frac{1,616.65}{117.05} = 13.81\ in.$ -13.81x1,616.65 = - 22,329

I(Neutral Axis) = 25,555 in.^{4}

d(top of steel) = 17.78 – 13.81 = 3.97 in.

d(bot. of steel) = 17.78 + 0.375 + 13.81 = 31.97 in.

S(top of steel) = $\frac{25,555}{3.97} = 6,437\ in.^3$

S(bot. of steel) = $\frac{25,555}{31.97} = 799\ in.^3$

d(top of conc.) = 26.41 – 13.81 = 12.60 in.

S(top of conc.) = $\frac{25,555}{12.60} = 2,028\ in.^3$

Since the compression flange is attached to the concrete slab by studs, the flange is assumed to be adequate for local buckling and compression flange bracing requirements.

Check Web for Buckling

$d/_{tw} \leq \frac{13,300}{F_y}$

$d/_{tw} = \frac{35.55}{0.598} = 59.45 \leq \frac{13,300}{36000} = 70.1$

The girder satisfies the requirements for a compact section. However, Wisconsin Bridge Office practice is to base moment strength for positive moment composite section on "non-Compact Section" criteria.

$M_{L+I} = S^*[F_y - 1.3(D1/SD_1\text{-}D2/SD_2)]$
n = 8 for live load
S = 799 = composite section modulus for bottom of girder

D1 = non-composite dead load moment

SD_1 = section modulus of steel

D2 = superimposed dead load moment on composite section

SD_2 = composite section modulus using n = 24

M_{L+I} = 799*[36.-1.3*309*12/536 – 1.3*57*12/723]

M_{L+I} = 799*[36-8.99 – 1.23]/12 = 1716 ft. kips

M_{L+I} = 1.3*RF*(L+I)

RF = 1716/(1.3*L+I)

RF = 1716/(1.3*750) = 1.76

Operating Rating = 1.76*HS20 = HS35

Inventory Rating = .6*1.76*HS20 = HS21

<u>Standard Permit Vehicle Rating</u>

From a computer run the maximum moment from at standard permit vehicle occurs at the .4 span point and equals 1224 ft. kips including impact. This moment is based on single lane distribution.

M_{L+I} = 1.3*RF*(L+I)

RF = 1716/(1.3*1224)

RF = 1.08

The moment from a 250 kips permit vehicle at the pier is –955 including impact.

M_{L+I} = 1.3*RF*(L+I)

RF = 1512/(1.3*955) = 1.22

Permit vehicle rating = 250 kips

Rating Based on Fatigue

The allowable stress range at the end of the 10 x 5/8" cover plate equals 13 ksi from Table 10.3.1A in AASHTO. This allowable is based on Category E and 500,000 cycles.

Dead load moment 4.75' from pier from computer output = -487 ft. kips.

Live load truck moment 4.75' from pier from computer output = -321. ft. kips.

S (top of steel) = 460.

Stress range from HS20 truck = M/S = 321*12/460 = 8.37 ksi

RF = 13/8.37 = 1.55

The stress range at the end of the 10 x 7/8 cover plate (.2 span point) must also be checked.

Dead load moment = 297 ft. kips

Live load truck moment = 588.4 ft. kips

S = 604 = composite section modulus for bottom flange.

Stress range from HS20 truck = M/S = 588*12/604 = 11.68 ksi

RF = 13/11.68 = 1.11

Inventory rating based on fatigue equals 1.11*HS20 = HS22

Summary

Inventory Rating ……….HS21
Operating Rating ………HS35

Standard Permit
Vehicle Rating ………..…250 kips

250 KIP STANDARD PERMIT VEHICLE LIVE LOAD MOMENTS ON LONGITUDINAL GIRDERS OF ONE SPAN

Live Load Moments in Foot-Kips per Wheel Line with Impact

Span Tenth Points

Span (Feet)	.1	.2	.3	.4	.5
24	116.4	201.8	266.2	305.0	312.0
28	151.5	255.7	327.8	381.4	390.0
32	179.1	308.6	394.7	445.2	468.0
36	207.3	356.8	464.5	530.8	546.0
40	234.3	412.1	533.3	605.7	624.0
44	263.9	463.5	603.4	682.0	699.8
48	296.0	511.6	668.2	763.1	777.6
52	326.7	571.2	735.7	828.5	853.2
56	369.5	614.6	800.9	914.8	935.8
60	412.8	690.2	867.9	981.3	1008.3
64	457.8	778.6	962.4	1055.1	1089.1
68	509.9	866.4	1078.2	1159.2	1169.4
72	563.3	953.5	1193.8	1273.6	1255.4
76	615.9	1045.8	1308.6	1408.0	1359.6
80	667.0	1141.3	1435.2	1554.9	1481.8
84	729.6	1247.3	1567.3	1700.8	1621.8
88	778.2	1331.7	1685.7	1832.4	1767.2
92	825.3	1426.7	1802.2	1976.0	1911.8
96	885.7	1521.2	1915.1	2118.9	2055.5
100	930.4	1610.3	2048.7	2261.1	2198.5
104	989.6	1703.8	2169.6	2402.6	2352.3
108	1032.1	1801.8	2295.9	2530.6	2498.7
112	1090.2	1880.0	2421.6	2662.3	2640.2
116	1133.5	1983.3	2538.4	2805.6	2781.1
120	1187.5	2068.0	2650.5	2944.0	2921.4
124	1244.3	2151.2	2762.1	3081.9	3061.1
128	1283.0	2252.1	2873.4	3219.2	3200.4
132	1337.6	2345.6	2986.1	3356.1	3339.1
136	1393.2	2413.5	3118.0	3492.5	3485.0

250 KIP STANDARD PERMIT VEHICLE LIVE LOAD MOMENTS ON LONGITUDINAL GIRDERS OF TWO EQUAL LENGTH SPANS CONSTANT MOMENT OF INERTIA

Live Load Moments in Foot-Kips per Wheel Line with Impact

Span Tenth Points

Span (Feet)	.3	.4	.5	.6	.7	1.0
24	224.6	241.1	237.8	213.6	158.4	-178.5
28	278.0	310.0	300.7	262.5	197.7	-250.9
32	338.0	365.6	360.3	315.9	242.7	-355.3
36	402.3	436.4	428.0	378.7	288.6	-445.5
40	459.3	493.5	487.3	435.9	335.4	-540.1
44	522.1	562.0	545.5	484.7	381.4	-622.2
48	581.7	631.2	610.1	532.6	423.0	-692.0
52	637.3	684.9	665.9	593.0	461.7	-752.5
56	700.1	759.3	730.9	649.8	506.4	-805.7
60	755.9	811.2	789.0	696.3	557.3	-853.7
64	807.2	879.5	853.0	756.3	614.9	-894.4
68	902.9	950.6	916.9	838.8	680.5	-929.3
72	988.1	1038.1	980.3	913.4	746.4	-964.1
76	1089.7	1140.7	1057.7	1007.6	813.0	-989.5
80	1196.5	1247.0	1148.8	1098.0	883.6	-1020.0
84	1305.3	1356.5	1253.4	1203.7	961.2	-1042.7
88	1403.9	1467.6	1357.9	1304.2	1028.7	-1064.8
92	1499.6	1577.0	1462.2	1400.6	1101.6	-1085.0
96	1595.5	1689.1	1582.5	1497.5	1166.5	-1097.5
100	1706.5	1802.1	1704.2	1594.5	1250.3	-1147.5
104	1810.4	1916.8	1825.8	1692.0	1322.7	-1214.5
108	1921.0	2019.7	1934.5	1786.8	1402.3	-1277.4
112	2032.0	2125.6	2038.9	1895.5	1482.7	-1341.8
116	2134.9	2242.8	2143.5	1996.2	1555.5	-1407.0
120	2234.0	2356.6	2253.0	2092.7	1625.0	-1468.6
124	2333.3	2470.6	2370.6	2189.7	1694.9	-1528.2
128	2432.6	2585.0	2488.0	2286.6	1765.3	-1589.4
132	2532.4	2700.1	2605.4	2384.0	1836.6	-1651.8
136	2632.1	2815.4	2722.7	2481.4	1919.4	-1712.4

250 KIP STANDARD PERMIT VEHICLE LIVE LOAD MOMENTS ON LONGITUDINAL GIRDERS OF THREE EQUAL LENGTH SPANS CONSTANT MOMENT OF INERTIA

Live Load Moments in Foot-Kips per Wheel Line with Impact

Span Tenth Points

Span (Feet)	Span No.	0.0/1.0	0.1/0.9	0.2/0.8	0.3/0.7	0.4/0.6	0.5
24	1	0.0	105.1	176.7	225.6	253.2	247.0
		40.6	36.6	114.9	179.9	226.9	
	1	0.0	-12.3	-24.7	-37.0	-49.4	-61.7
		-183.3	-111.1	-98.8	-86.4	-74.1	
	2	40.6	20.3	92.1	149.0	184.1	192.4
		-183.3	-142.2	-121.9	-117.0	-112.7	-110.1
28	1	0.0	137.5	224.1	275.0	305.2	294.8
		49.6	44.6	137.0	210.5	259.2	
	1	0.0	-13.4	-26.7	-40.1	-53.5	-66.9
		-224.5	-155.8	-120.9	-99.3	-80.2	
	2	49.6	31.4	114.4	182.2	217.4229.2	
		-224.5	-177.1	-148.7	-129.0	-117.3	-112.5
32	1	0.0	162.8	269.9	334.4	360.4353.1	
		57.9	52.1	134.9	232.5	307.3	
	1	0.0	-13.9	-27.7	-41.6	-55.4	-69.3
		-320.8	-218.2	-144.3	-97.8	-83.2	
	2	57.9	53.1	135.4	199.8	236.3248.9	
		-320.8	-233.3	-173.7	-144.8	-115.8	-93.0
36	1	0.0	188.0	314.0	398.2	430.1420.2	
		66.7	60.0	153.9	277.1	368.7	
	1	0.0	-14.9	-29.7	-44.6	-59.5	-74.3
		-415.1	-271.1	-158.2	-104.1	-89.2	
	2	66.7	68.6	170.8	240.8	270.1270.6	
		-415.1	-285.6	-200.1	-166.8	-133.4	-100.1
40	1	0.0	212.8	363.6	454.3	486.6478.2	
		75.4	67.9	181.8	322.5	424.6	
	1	0.0	-15.8	-31.7	-47.5	-63.3	-79.2
		-503.7	-321.1	-170.8	-110.8	-95.0	
	2	75.4	77.3	197.1	271.7	306.8302.0	
		-503.7	-332.8	-226.4	-188.6	-150.9	-113.2
44	1	0.0	240.6	410.8	516.7	554.2	535.3
		83.5	75.1	209.2	367.2	472.2	
	1	0.0	-17.7	-35.5	-53.2	-70.9	-88.7
		-583.0	-363.1	-176.3	-124.1	-106.4	
	2	83.5	84.0	229.8	315.9	346.3	329.6
		-583.0	-374.6	-250.4	-208.7	-167.0	-125.2

250 KIP STANDARD PERMIT VEHICLE LIVE LOAD MOMENTS ON
LONGITUDINAL GIRDERS OF THREE EQUAL LENGTH SPANS
CONSTANT MOMENT OF INERTIA

Live Load Moments in Foot-Kips per Wheel Line with Impact

Span Tenth Points

Span (Feet)	Span No.	0.0/1.0	0.1/0.9	0.2/0.8	0.3/0.7	0.4/0.6	0.5
48	1	0.0	272.1	456.0	576.0	622.4	599.2
		91.5	82.4	236.0	407.7	519.0	
	1	0.0	-20.6	-41.2	-61.8	-82.4	-103.0
		-653.6	-397.3	-173.6	-144.2	-123.6	
	2	91.5	82.6	254.8	354.4	391.3	378.1
		-653.6	-406.0	-274.5	-228.8	-183.0	-137.3
52	1	0.0	302.9	507.6	630.8	675.5	653.8
		100.3	90.3	262.5	445.3	578.2	
	1	0.0	-24.5	-48.9	-73.4	-97.8	-122.3
		-716.1	-424.0	-195.7	-171.2	-146.7	
	2	100.3	80.1	285.0	400.8	443.4	422.0
		-716.1	-430.7	-301.0	-250.8	-200.6	-150.5
56	1	0.0	320.8	550.6	693.3	748.9	717.2
		109.3	98.4	288.6	488.2	633.8	
	1	0.0	-29.2	-58.5	-87.7	-116.9	-146.1
		-770.6	-443.5	-233.8	-204.6	-175.4	
	2	109.3	73.9	318.1	458.4	514.7	492.5
		-770.6	-449.4	-327.9	-273.2	-218.6	-163.9
60	1	0.0	369.1	600.8	748.5	799.8	774.4
		118.2	106.4	314.5	537.0	679.2	
	1	0.0	-34.7	-69.3	-104.0	-138.2	-173.3
		-818.9	-457.4	-277.3	-242.6	-207.6	
	2	118.2	69.6	341.8	504.8	572.8	553.0
		-818.9	-463.4	-354.7	-295.6	-236.5	-177.4
64	1	0.0	402.7	665.0	799.0	867.7	877.3
		137.0	123.3	340.2	591.7	737.2	
	1	0.0	-40.3	-80.7	-121.0	-161.3	-201.7
		-860.9	-467.6	-322.7	-282.3	-242.0	
	2	137.0	68.5	368.4	561.7	645.0	619.3
		-860.9	-479.6	-411.1	-342.5	-247.0	-205.5
68	1	0.0	449.1	742.0	892.0	938.0	900.1
		155.3	139.8	369.6	654.0	817.4	
	1	0.0	-46.3	-92.5	-138.8	-185.0	-231.3
		-902.2	-475.3	-370.0	-323.8	-277.5	
	2	155.3	77.6	402.4	620.1	728.8	703.1
		-902.2	-543.5	-465.9	-388.2	-310.6	-232.9

250 KIP STANDARD PERMIT VEHICLE LIVE LOAD MOMENTS ON LONGITUDINAL GIRDERS OF THREE EQUAL LENGTH SPANS CONSTANT MOMENT OF INERTIA

Live Load Moments in Foot-Kips per Wheel Line with Impact

Span Tenth Points

Span (Feet)	Span No.	0.0/1.0	0.1/0.9	0.2/0.8	0.3/0.7	0.4/0.6	0.5
72	1	0.0	494.9	817.8	976.6	1023.3	962.0
		173.5	156.2	400.5	716.6	889.4	
	1	0.0	-52.2	-104.3	-156.5	-208.7	-260.8
		-933.4	-474.0	-417.3	-365.2	-313.0	
	2	173.5	86.8	435.0	677.7	796.4	776.5
		-933.4	-607.3	-520.5	-433.8	-347.0	-260.3
76	1	0.0	540.5	892.6	1075.5	1122.8	1037.6
		193.0	173.7	428.3	780.0	981.5	
	1	0.0	-58.0	-116.1	-174.1	-232.2	-290.2
		-966.0	-522.4	-464.4	-406.3	-348.3	
	2	193.0	96.5	466.9	738.8	868.4	855.8
		-966.0	-675.5	-579.0	-482.5	-386.0	-289.5
80	1	0.0	585.8	972.2	1180.6	1226.5	1126.6
		212.5	191.2	461.4	846.8	1067.6	
	1	0.0	-63.8	-127.7	-191.5	-255.4	-319.2
		-991.8	-574.6	-510.8	-446.9	-383.1	
	2	212.5	106.2	503.9	803.8	948.1	939.3
		-991.8	-743.6	-637.4	-531.1	-424.9	-318.7
84	1	0.0	639.6	1062.6	1287.8	1333.5	1228.9
		232.9	209.6	502.0	920.8	1171.7	
	1	0.0	-70.1	-140.2	-210.2	-280.3	-350.4
		-1122.9	-630.7	-560.6	-490.5	-420.5	
	2	232.9	116.5	548.5	873.6	1038.0	1026.5
		-1022.9	-815.2	-698.8	-582.3	-465.8	-349.4
88	1	0.0	684.3	1135.2	1385.1	1443.2	1330.6
		250.5	225.5	530.0	984.9	1269.0	
	1	0.0	-75.7	-151.3	-227.0	-302.6	-378.3
		-1041.4	-680.9	-605.2	-529.6	-453.9	
	2	250.5	125.3	580.0	934.8	1119.7	1106.2
		-1041.4	-876.9	-751.6	-626.3	-501.1	-375.8
92	1	0.0	729.4	1214.1	1479.4	1550.4	1433.0
		270.2	243.2	562.0	1054.9	1362.2	
	1	0.0	-81.5	-163.0	-244.5	-326.0	-407.5
		-1080.8	-733.6	-652.1	-570.5	-489.0	
	2	270.2	135.1	616.0	1000.4	1198.5	1193.3
		-1080.8	-945.7	-810.6	-675.5	-540.4	-405.3

250 KIP STANDARD PERMIT VEHICLE LIVE LOAD MOMENTS ON
LONGITUDINAL GIRDERS OF THREE EQUAL LENGTH SPANS
CONSTANT MOMENT OF INERTIA

Live Load Moments in Foot-Kips per Wheel Line with Impact

Span Tenth Points

Span (Feet)	Span No.	0.0/1.0	0.1/0.9	0.2/0.8	0.3/0.7	0.4/0.6	0.5
96	1	0.0	781.4	1300.2	1574.4	1660.4	1552.6
		287.7	258.9	604.1	1116.6	1456.1	
	1	0.0	-87.0	-174.0	-261.1	-348.1	-435.1
		-1150.9	-783.2	-696.2	-609.1	-522.1	
	2	287.7	143.9	661.4	1058.3	1280.0	1284.0
		-1150.9	-1007.0	-863.2	-719.3	-575.4	-431.6
100	1	0.0	826.3	1373.3	1683.7	1771.5	1671.7
		306.0	275.4	639.7	1197.0	1550.0	
	1	0.0	-92.8	-185.5	-278.3	-371.1	-463.9
		-1224.0	-835.0	-742.2	-649.4	-556.6	
	2	306.0	153.0	699.6	1136.2	1362.5	1376.0
		-1224.0	-1071.0	-918.0	-765.0	-612.0	-459.0
104	1	0.0	877.3	1462.5	1786.5	1884.4	1790.7
		323.9	291.5	679.6	1266.3	1644.6	
	1	0.0	-98.0	-195.9	-293.9	-391.8	-489.8
		-1295.4	-881.6	-783.7	-685.7	-587.8	
	2	323.8	161.9	743.6	1203.4	1447.0	1470.5
		-1295.4	-1133.4	-971.5	-809.6	-647.7	-485.8
108	1	0.0	922.4	1544.7	1896.0	1985.6	1896.9
		340.6	306.6	715.5	1342.7	1738.6	
	1	0.0	-103.7	-207.3	-311.0	-414.6	-518.3
		-1362.5	-932.9	-829.3	-725.6	-622.0	
	2	340.6	170.3	783.4	1278.2	1527.6	1553.8
		-1362.6	-1192.2	-1021.9	-851.6	-681.3	-511.0
112	1	0.0	972.6	1623.5	2006.0	2089.8	1998.9
		357.8	322.0	760.7	1420.1	1844.4	
	1	0.0	-109.0	-218.1	-327.1	-436.2	-545.2
		-1431.2	-981.4	-872.4	-763.3	-654.3	
	2	357.8	178.9	830.7	1354.2	1622.9	1634.5
		-1431.2	-1252.3	-1073.4	-894.5	-715.6	-536.7
116	1	0.0	1008.2	1711.2	2108.0	2205.3	2101.0
		375.2	337.7	800.9	1490.0	1942.2	
	1	0.0	-114.1	-228.2	-342.3	-456.4	-570.5
		-1500.7	-1026.9	-912.8	-798.7	-684.6	
	2	375.2	187.6	874.7	1423.0	1710.9	1722.5
		-1500.7	-1313.1	-1125.5	-937.9	-750.4	-562.8

250 KIP STANDARD PERMIT VEHICLE LIVE LOAD MOMENTS ON
LONGITUDINAL GIRDERS OF THREE EQUAL LENGTH SPANS
CONSTANT MOMENT OF INERTIA

Live Load Moments in Foot-Kips per Wheel Line with Impact

Span Tenth Points

Span (Feet)	Span No.	0.0/1.0	0.1/0.9	0.2/0.8	0.3/0.7	0.4/0.6	0.5
120	1	0.0	1067.8	1780.8	2206.3	2317.4	2211.4
		391.6	352.5	836.0	1556.6	2036.2	
	1	0.0	-119.4	-238.8	-358.2	-477.6	-596.9
		-1566.4	-1074.5	-955.1	-835.7	-716.3	
	2	391.6	195.8	909.2	1488.7	1795.9	1816.3
		-1566.5	-1370.6	-1174.8	-979.0	-783.2	-587.4
124	1	0.0	1117.2	1870.2	2304.7	2429.8	2326.7
		407.5	366.8	889.0	1623.7	2130.2	
	1	0.0	-124.8	-249.5	-374.3	-499.1	-623.8
		-1630.1	-1122.9	-998.1	-873.4	-748.6	
	2	407.5	203.8	967.8	1555.0	1881.3	1910.6
		-1630.1	-1426.3	-1222.5	-1018.8	-815.0	-611.3
128	1	0.0	1148.4	1956.1	2403.3	2542.8	2441.7
		423.8	381.5	927.4	1691.4	2224.4	
	1	0.0	-129.9	-259.8	-389.7	-519.6	-649.5
		-1695.4	-1169.1	-1039.2	-909.3	-779.4	
	2	423.8	211.9	1010.2	1622.1	1967.4	2005.8
		-1695.4	-1483.5	-1271.6	-1059.6	-847.7	-635.8
132	1	0.0	1212.0	2034.8	2502.3	2656.4	2556.9
		440.5	396.4	963.1	1760.1	2319.1	
	1	0.0	-134.7	-269.5	-404.2	-538.9	-673.7
		-1761.9	-1212.6	-1077.9	-943.2	-808.4	
	2	440.5	220.2	1047.7	1690.4	2054.7	2102.2
		-1761.9	-1541.6	-1321.4	-1101.2	-880.9	-660.7
136	1	0.0	1260.6	2103.5	2601.3	2770.3	2671.9
		456.6	411.0	1017.6	1839.5	2414.0	
	1	0.0	-139.7	-279.3	-419.0	-558.6	-698.3
		-1826.5	-1256.9	-1117.2	-977.6	-837.9	
	2	456.6	228.3	1104.1	1760.2	2142.4	2199.2
		-1826.6	-1598.2	-1369.9	-1141.6	-913.3	-685.0

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